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The Spokane River and Hangman Creek Viaduct of the O. W. R. R. & N. Co.

By J. O. Baar*

Where the Oregon Washington R.R. & Navigation Co. gains entrance to the City of Spokane, Wash., over its new line from the west, it crosses the valleys of the Spokane River and Hangman Creek on a single-track steel viaduct 3003 ft. long and 187 ft. high above low water. At the site of the bridge, the Hangman or Latah Creek is nearing the end of its course, and is separated from the Spokane River, which it parallels at that point, by a low ridge only a few hundred feet wide. The bridge traverses the valleys at right angles and at the narrowest point.

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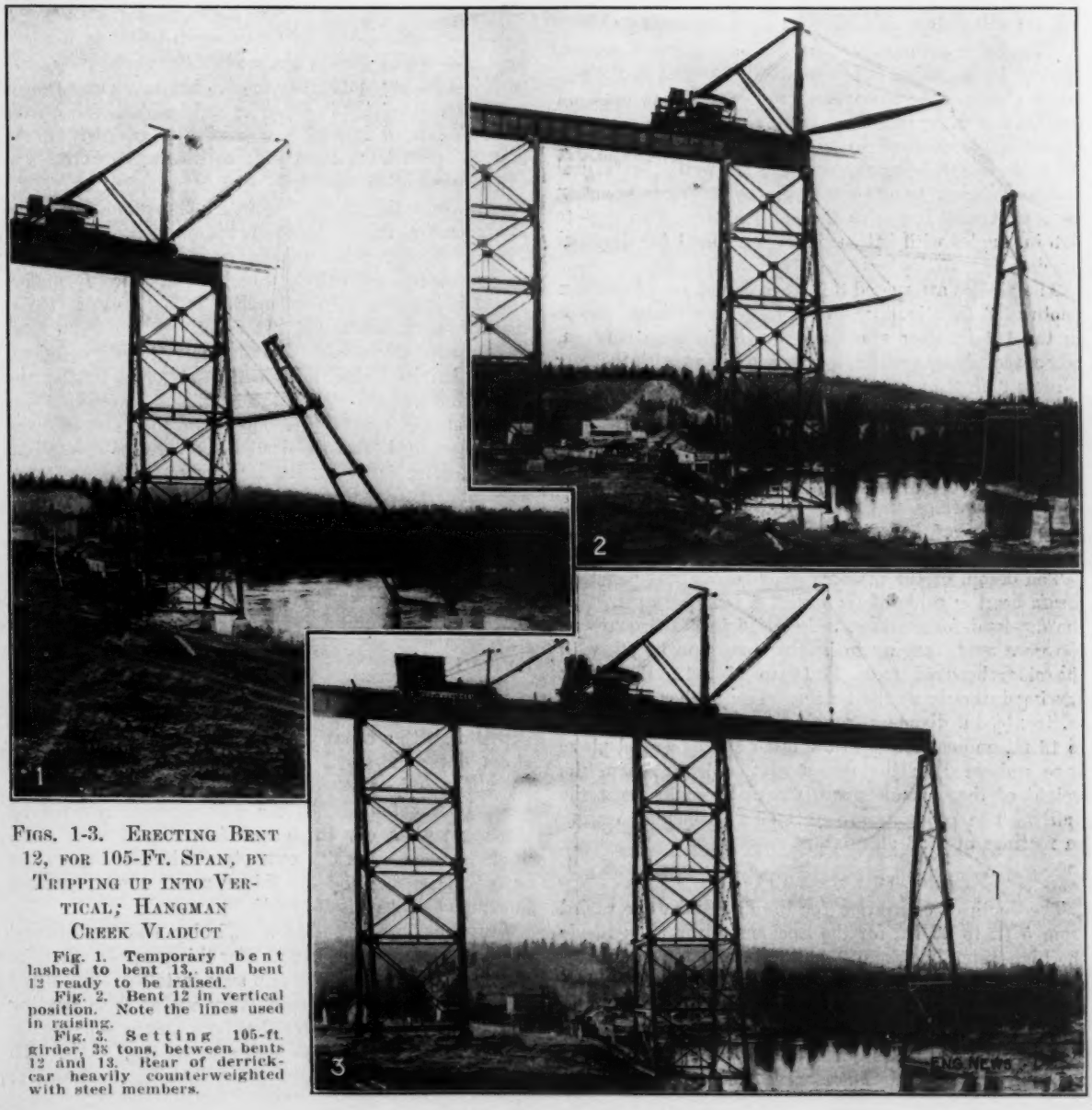

Figs. 1-3. Erecting Bent 12, for 105-Ft. Span, by Tripping up into Vertical; Hangman Creek Viaduct

Fig. 1. Temporary bent lashed to bent 13, and bent 12 ready to be raised.

Fig. 2. Bent 12 in vertical position. Note the lines used in raising.

Fig. 3. Setting 105-ft. girder, 38 tons, between bents 12 and 13. Rear of derrick-car heavily counterweighted with steel members.

Description of the Viaduct

Design—The alignment of the bridge is tangent for 2700 ft., terminating in a spiraled 5° curve on the west end. The grade is 0.6% compensated, downward to the east.

Several preliminary studies were made in connection with this structure. The design finally decided upon contains two spans of 105 ft., 17 of 80 ft., four of 70 ft., one of 60 ft., three of 50 ft., 20 of 40 ft., one of 38 ft. and three of 35 ft., all deck plate-girder spans. The main portion consists of 80-ft. spans between 40-ft. towers, with two 105-ft. river spans.

The design of the superstructure was based on the O. W. R.R. & N. standard wheel-loading, which practically corresponds to Cooper's E55. The specifications used allowed a stress of 8000 $\left(1+\frac{min.}{max.}\right)$, reduced by 35 l/r for compression. A rather low bearing value on concrete masonry was used, 250 lb. per sq.in. This necessitated the use of large cast-steel column bases, those for the river piers being 4 ft. 2 in. square.

The tower columns are of built-up channel section, having 18-in. webs, 4x4- and 6x6-in. angles and 22-in. cover plates. The maximum cross-section (at bents 11 and 12) is 67 sq.in. The struts are made up of 12-in. channels placed back to back and latticed, those over 25 ft. in length being supported in the center by vertical hangers riveted to the gusset-plates at the intersection of the diagonal braces in the panel above. Two 4x3- to 5x3-in. angles with batten plates were used for diagonal bracing.

The girders are spaced 8 ft. c. to c. and are of uniform depth of 8 ft. 9 in. throughout the entire bridge, except in the 105-ft. river spans and the 50-ft. span between, where they are spaced 10 ft. center to center, with the long girders 9 ft. 8 in. deep. They were designed with *clear top flanges* built up of 6x4-in. angles and 8-in. side plates, allowing all ties to be of the same depth and without daps for cover plates or rivet heads.

All 70-ft. and 80-ft. girders were designed with simple sliding expansions at the down-grade end. The 105-ft. girders, however, have rocker bearings and segmental expansion rollers working between machined surfaces.

The design of the masonry foundations was based on a safe bearing value of the soil of 4 tons per sq.ft. For the dry-land foundations, pedestals 5 to 6 ft. square on top were used, ranging in height from 6 to 24 ft., with the sides battered from 1 : 12 to 2½ : 12, the batter varying inversely as the height. Two anchor bolts of 2 in. to 2½ in. diameter, ranging in length from 7½ ft. to 13 ft., and connected at the lower end by a steel plate, were embedded in the masonry at a depth where the weight of the concrete above the plate would resist the uplift. The pedestals are of 1 : 3 : 5 concrete, resting on footings of 1 : 3 : 6 mixture.

Open-Caisson Piers

The depth of excavation for the dry-land work varied from 5 ft. to 32 ft., for the Spokane River piers inside the high-water line from 9 to 20 ft., and for the Hangman Creek piers 14 to 21 ft. All dry excavation was done by hand, a force of from 10 to 25 men keeping this work well in advance of the concreting crew.

The Spokane River piers were constructed by the open-caisson process. These piers are diamond shaped, 6 ft. wide on top and 12 ft. from point to point, with the sides battered 1¼ : 12. They rest on from 22 to 26 timber piles.

The formation quite uniformly encountered was a deep bed of well packed coarse gravel and sand with embedded boulders, overlaid with from 2 to 4 ft. of sandy soil. Occasionally, pockets of volcanic ash were met with. Test-borings carried to considerable depth in the river beds revealed no change in the general formation.

For the river work, a temporary pile trestle was first built on the center-line of the bridge, with the deck at an elevation of 8 ft. above low water, which was sufficiently high to take care of such floods as might reasonably be expected during the period of construction. Extensions of this trestle at right angles were built at the center of the river tower, so that all piers could be reached. The caissons, averaging 22 ft. in depth, with inside dimensions 2 ft. larger all around than the footings of the piers, were built up of 3x10-in. plank laid on the 10-in. face and spiked together with 60d. nails. Sections 6 ft. high were put together on the trestle, a V-shaped cutting edge of timber drift-bolted to the bottom, and towed out to their location, where they were anchored to guy piles, previously driven. The caissons were then completed in the river and weighted down with rails distributed in such a manner that any part of the interior could be reached with an excavating bucket.

The excavation was done with a clamshell bucket of 1 cu.yd. capacity, operated from a stiffleg figure-four derrick placed on the trestle at the intersection of the wings. The excavated material was deposited around the caissons. In excavating, the cutting edge was first evenly undermined to insure a uniform settling of the caisson. Occasionally, a caisson would list rapidly to one side; then the opposite side would be additionally counter-weighted and undermined until it righted itself again. After the excavation was completed, 4x8-in. t. & g. sheetpiling was driven snugly against the outside of the caisson, from leads suspended from the derrick. Where the sheetpiling did not drive up tight, the intervening space was filled with cinders, rammed to shut out the water.

The foundation piles, 22 to 26 under each pier, were driven next, with a 2200-lb. drophammer piledriver equipped with 30-ft. leads and 15-ft. extension leads, and 7x10-in. hoisting engine using a ⅝-in. steel cable for the hammer line and a ⅜-in. steel cable for the pile line. The piles were driven to refusal at an average penetration of 18 ft.

Before attempting to pump the caissons dry, the bottom was sealed with a 4-ft. layer of 1 : 3 : 5 concrete deposited with a tremie: a 10-in. steel pipe slightly flared at the lower end and a 36-in. receiving funnel riveted to the upper end, operated from a scaffold set on the caisson. It was found that the best concrete resulted by starting with a dry batch and following it with semi-fluid concrete. The dry concrete would force the water out of the tremie and the thin concrete would ooze out slowly and spread itself evenly.

Leakage Trouble—Much difficulty was encountered in pumping out the caissons. For this purpose, one 6-in. and one 8-in. centrifugal pump were installed. The many joints in the caissons, almost every one of them proving leaky, let in so much water that both pumps working together never lowered the water more than 7 ft. To make the joints water-tight, cedar shingles sharpened to

a thin edge were driven into the larger cracks of the pumped-out section, after which the water could be lowered a few feet farther, and the process repeated; afterwards, the small leaks were stopped with waste. The piles were then cut off 2 ft. above the concrete, and the remaining portion of the footings completed with 1 : 3 : 5 concrete.

For the construction of the Hangman Creek piers, it was not necessary to build a temporary trestle. At the time the work was commenced, the creek was practically dry. However, Hangman Creek is subject to severe and sudden floods, so it was decided to use caissons for the

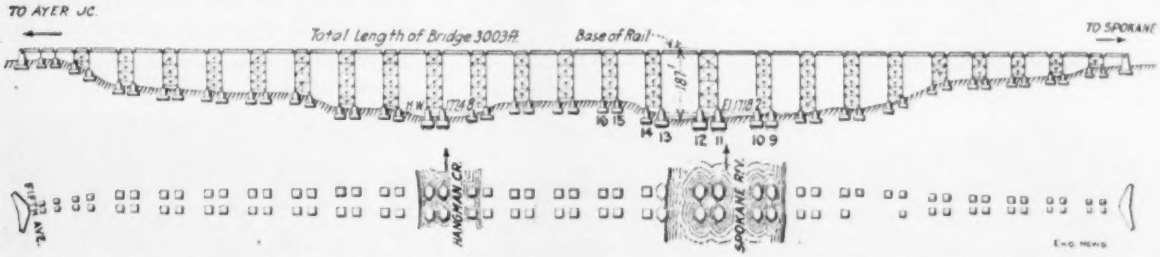

FIG. 4. DIAGRAM OF THE O. W. R.R. & N. CO.'S VIADUCT OVER SPOKANE RIVER AND HANGMAN CREEK AT SPOKANE, WASH.

six piers, similar to those employed on the Spokane River foundations. It was well that such provision had been made; the creek rose rapidly while excavating, but no damage was done to the foundations. The work of excavating was done by hand, using buckets operated from a small hoist to dispose of the muck. The foundation piles were driven and cut off before placing concrete, one 6-in. centrifugal pump keeping the pits dry.

The forms for piers and pedestals were built of 2x8-in. t. & g. fir and tamarack, with four 2x8-in. studs on the north and south faces and three 2x8-in. studs on the others. The two center studs on the north and south sides were spaced 6 in. center to center and cut off 2 ft. above the top of masonry. These were used to support a 4x8-in. plate, provided with two 2-in. or 2½-in. holes through which the anchor-bolts were hung. Around the anchor-bolts were placed tapered boxes made of half-inch lumber, 4 ft. long, 6 in. square at the upper end and 3 in. square at the lower end, the top flush with the top of the pier. Before concreting, the boxes were covered with grease, and upon completion of the pedestal they were removed, thus leaving sufficient space to correctly realign anchor-bolts which might have been displaced. It was found best to loosen the boxes about one hour after the last of the concrete was poured, and then no difficulty was experienced in removing them entirely when the forms were taken down. Occasionally, this would be neglected and the boxes were so tight that they had to be burned out.

Perhaps 60% of the concrete was placed in freezing weather, with temperatures as low as 18° below zero; then proper precautions had to be taken to insure satisfactory results. The sand and crushed rock were heated by turning steam into perforated wooden boxes 8 in. square and 20 ft. long, on top of which the aggregate was dumped to a depth of 3 ft. directly from the wagons. This proved sufficient at all times to keep the frost out of the material. The water was heated to a temperature of about 70° and the exhaust steam of the mixer engine was discharged into the drum. Canvas placed over the top of the forms and one or two lanterns in the space between the canvas and the concrete kept the frost out successfully. The cost of heating the aggregate, etc., was 12½c. per cu.yd. concrete.

ERECTING THE SUPERSTRUCTURE

The steel for the superstructure, shipped from the East to Hooper, on the O. W. R.R. & N. Co.'s cutoff, about 83 miles southwest of Spokane, was brought to the site over the new line. The west approach of the bridge being an embankment over 3000 ft. long and an average of 20 ft. high, followed by a series of short and comparatively high cuts and fills, the storage facilities were not the best. The steel was therefore distributed along the roadbed for a mile and a half west of the bridge. A blind siding in a borrowpit served to store and switch derrick cars and steel cars awaiting unloading. For unloading, two small self-propelling derrick cars, equipped with a two-drum, three-spool hoisting engine, 40-ft. timber boom hinged to a revolving center post in the A-frame, rigged for a capacity of 25 tons, were used. For the erection, a stiffleg A-frame derrick-car was specially designed. The body of the car, consisting of six 9x18-in. by 70-ft. timbers, cross-braced and so spaced that the width of the car was made 11 ft., was carried on one eight-wheeled truck in front and two sets of wheels placed 6 ft. and 20 ft., respectively, from the rear end of the car. The derrick was run on four rails spaced standard and 8-ft. gage. The A-frame, 50 ft. high, 5 ft. wide on top and 11 ft. at the base, was framed of two 12x14-in. batter posts and one 18x19-in. center post, cross-braced in the manner of a trestle bent. Two rear stifflegs of 14x14-in. by 80-ft. timbers extended from the top of A-frame to the rear of the car. The boom consisted of one 18x18-in. by 81-ft. Oregon fir timber, trussed on all four sides with a 4x18-in. plank. The fixed end was hinged to a bullwheel placed directly in front of the A-frame, a special engine operating the swing cable. Power was supplied by a double-drum, four-spool American hoisting engine. The drums carried ⅝-in. steel cable rove through six-sheave blocks on the mast and boom. From the spools were operated two runners or side lines running through single sheaves placed at the end of the boom, one on each side of the load line.

This car never left the bridge. The steel was brought to it by the small derrick cars and placed on outriggers consisting of 12x12 timbers 22 ft. long, put under the track and projecting 6 ft. beyond the ends of ties, braced by struts resting on the bottom flanges of the girders. As the erection progressed, these outriggers were moved ahead so that the main car never had to back up more than 500 ft. to get the material.

Before placing the cast column bases, the piers were carefully gone over, and, where the shrinkage of the concrete was less than provided for, bush-hammered to the proper elevation. The work of erection was carried on from the west end only. After the bents reached a certain height, it was found most economical to bolt and fit up two column sections in the yard, thereby lessening the work at the front. The columns and struts were placed with the load wire, the transverse and longitudinal braces with the side lines or runners. Where the ground was suited, the top section would be completely assembled on the ground and then raised. Temporary erection struts were used to stay the bents forming the east supports of open spans. The longitudinal or tower bracing was filled in as the erection of the farther bent progressed.

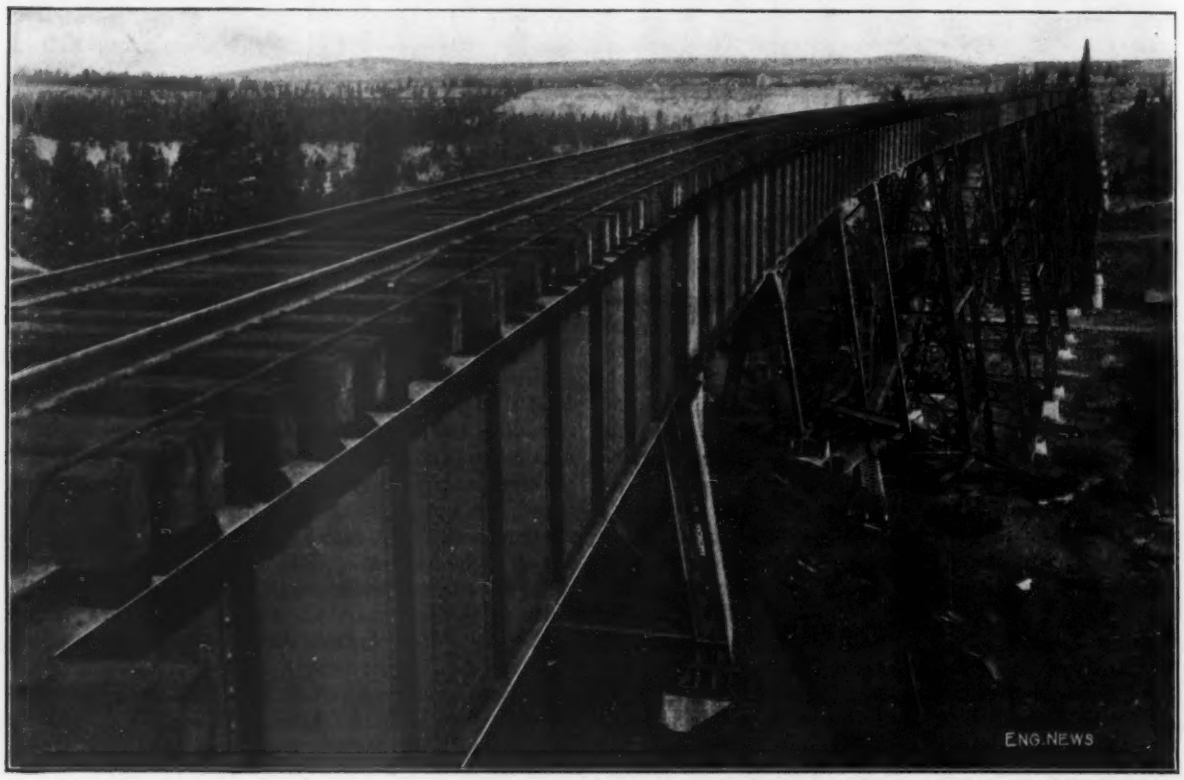

FIG. 5. THE COMPLETED SPOKANE-HANGMAN VIADUCT

ERECTING THE RIVER TOWER

The most interesting feature was the erection of bents 10 and 12, forming the east supports for the 105-ft. girders. With the equipment on hand, two methods were open: (1) to erect the bent vertical, necessitating the outhauling of every member a distance of 30 ft., or (2) to erect it in an inclined position on a temporary support. The latter method was employed.

For this purpose, a temporary timber bent 42 ft. high, consisting of two 12x14 posts and caps braced with 4x12 timbers, was framed. The center-to-center distance of the posts on one end corresponded with the c.-to-c. distance of the columns of bent 13 at a point just below the fourth transverse strut, and on the other end with the column spacing of bent 12 at the third transverse strut. One end of the bent was lashed to the columns of bent 13. The bent was held in a position slightly inclined from the horizontal by two lines fastened to bent 13 at the second strut from the top (see Fig. 2). The three lower column sections of the bent to be erected were completely riveted in the yard and the cast bases bolted on. A temporary timber blocking was put around the anchor-bolts in the piers, and the columns were so placed thereon that when the bent was vertical, the anchor-bolts would match the corresponding holes in the column bases.

These preparations being completed, the columns were moved to the outriggers. In placing these long members, the movement of the lower end had to be guided by one of the small derrick-cars until the column was in a vertical position, after which it had to be outhauled to the pier, and was finally placed with the upper end resting on the temporary bent. After both columns were properly aligned, the transverse struts were connected up, the bottom section was assembled on the ground and hoisted into place.

TRIPPING-UP THE BENT

With the bent in the inclined position, a shackle-chain was swung around the columns immediately above the third transverse strut, and to this one of a set of 12-in. double blocks was attached, the other block being made fast to a guy line carried out to a pier on the east shore of the river. The line rove through the set of blocks described above was carried upward and over the top of the bent and thence to a winch-head or spool on the derrick-engine. In order to keep this line from fouling the top strut, the bight was passed through a snatchblock made fast to the end of one of the runners. This block

is seen in the illustration immediately back of the diamond block at the end of the boom. The second runner line passed from its sheave through a snatchblock made fast to the top transverse strut, thence it led and was made fast to a pier on the east bank of the river. In order to prevent the bent from falling forward as it assumed a vertical position, should anything happen to the load line, two snublines were attached to the columns of the bent being erected, leading to bent 13 and made fast thereto.

Power was applied simultaneously to the load line, the line leading from the set of blocks guyed to the east side of the river and to the second runner line described above. The bent then assumed a vertical position, resting on blocking placed around the anchor-bolts, as previously described. With the bent in this position, a slight strain was taken on all the lines, holding the bent as rigidly as possible, while two 35-ton Norton jacks were placed under the lower transverse strut immediately adjacent to the columns, and the entire bent was lifted sufficiently to release the blocking. The blocking was then removed and the bent uniformly lowered over the anchor-bolts. The outhaul line was snubbed around the top transverse strut, and all other lines attached to the derrick car let go.

In the meantime, the 105-ft. girders were placed on the outriggers by the small derrick cars working boom to boom. The main car then backed up, and, picking up one of the girders, moved forward and landed it in position, after which it was bolted down. The other girder was then placed and the braces filled in.

Bent 10, the east support of the other 105-ft. span, was erected in a similar manner.

The riveting was done by compressed air, a plant of 250 cu.ft. per min. capacity being installed for that purpose at the west end of the bridge, from which a pipe line was extended to the front on the deck of the structure. From six to eight riveting crews riveted the steel as fast as it was erected. All field rivets were given one coat of black graphite paint.

Engineering

No difficult features presented themselves in establishing the center line. After the line was definitely located, substantial hubs were driven at intervals not exceeding 100 ft., projecting far enough above the ground surface to allow the tape to be suspended between points. In these, wire brads were set to serve as chaining points, the tops of hubs being trimmed to an apex at the brad. These hubs were numbered in succession from the east end, and the distance between them measured with a standard tested tape, graduated to feet and hundredths. Readings were taken to the nearest thousandth of a foot, the pull being measured with spring balances and the temperature read to the nearest half degree. Levels were then run carefully over the hubs, and, with the measured distance and the difference of elevation between points, the horizontal distance was computed, and checked in the field by horizontal chaining. At the center line of each bent, a substantial hub was driven 3 ft. into the ground, with the top flush with the surface. At every fourth bent, a concrete hub was placed. The Spokane River was triangulated first, and after the temporary trestle was built, the distance was checked by actual chaining. Great care was taken in setting the anchor-bolts, with very gratifying results throughout. After the bridge was completed, its length was measured on the deck, and checking the original chaining within 0.02 ft.

The construction of the substructure for this bridge was commenced in August, 1911, and rushed to completion by March, 1912. The quantities involved were: 8250 cu.yd. excavation, 14,400 lin.ft. piling, 1110 cu.yd. 1 : 3 : 6 concrete, 4155 cu.yd. 1 : 3 : 5 concrete, and 285 cu.yd. 1 : 2 : 4 concrete. For the superstructure, 3822 tons of structural steel were used.

A change in the plans for delivery of the steel caused the erection of the superstructure to be delayed until June, 1913. The bridge was completed in November, 1913, and opened for traffic the same month.

Chaugren, Boynton & Co., of Spokane, had the contract for the substructure. The superstructure was fabricated by the American Bridge Co. and erected by the Kelly-Atkinson Co., of Chicago. G. Schennewerk and E. Baggs acted as superintendents. The writer was engineer-in-charge for the Oregon-Washington R.R. & Navigation Co.

Permeability Tests on Gravel Concrete are being made at the University of Wisconsin. A progress report by Prof. Morton O. Withey, read before the Western Society of Engineers, Sept. 14, 1914, gave the following summary of the tests so far made.

1. None of the concretes tested were absolutely watertight if we consider continuous flow into the specimen as proof of permeability, but the majority of the mixes [varying from 1:1 to 1:3:6] were so impervious that no visible evidence of flow appeared. For most purposes such mixes can be considered water-tight.

2. The visibility of dampness on the bottom of the specimens increased with the humidity of the air and the non-homogeneity of the concrete. The minimum rate of flow for which leakage was indicated was 0.00011 gal. per sq.ft. per hr.

3. In tests of nearly all of the properly made mixes of 1:7 proportions, or richer, the rate of flow for a 50-hr. period was less than 0.0001 gal. per sq.ft. per hr. under a pressure of 40 lb. per sq.in.

4. Through increasing the fineness of the cement a reduction in the rate of flow and a considerable increase in the strength of a 1:9 mix were secured.

5. By grading the sand and gravel in accordance with Fuller's curve it was possible to obtain practically watertight concrete of 1:9 proportions under pressures less than 40 lb. per sq.in. To secure such results, however, requires great care and careful supervision in mixing, in determining the proper consistency, in placing, and in curing the concrete.

6. In the proportioning of such materials as these, volumetric analysis coupled with a determination of the density and air voids yields very valuable information concerning the best proportions of sand and gravel for a given proportion of cement. If proportions must be selected arbitrarily a 1:1½:3 mix, by volume, is very impervious.

7. The use of the proper amount of water necessary to produce a medium or mushy consistency is one of the most important conditions in securing impervious concrete, especially when lean mixtures are used. Dry mixtures cannot be sufficiently compacted in the molds and are more difficult to cure properly than the mushy mixtures. Although the use of a wet consistency does not materially affect the imperviousness of very rich mixes, such as 1:1½:3, it greatly increases the flow through a lean mix.

8. For lean mixes made from damp sand it seems advisable to mix longer than is now common practice. These tests would indicate that for a mixer running at 30 r.p.m., a period of one and one-half to two minutes is required to secure thorough mixing of a 1:9 concrete. For a rich 1:1½:3 mix a one-minute period appears to be sufficient. The method of mixing in which water is first admitted to the mixer is to be condemned. A preliminary period of dry mixing lasting from 15 to 30 sec. seems desirable.

9. No stage or process in the making of impervious concrete is of more importance than curing. The results of these tests clearly demonstrate that premature drying destroys the imperviousness of 1:9 mixes, seriously impairs that of the 1:2:4 mixes and somewhat diminishes that of the 1:1½:3 mixes. For thin sections, not over six or eight inches thick, the curing conditions should be such that a lean concrete will be kept damp for a period of one month and a rich concrete for at least two weeks. Even after a month of proper curing, complete desiccation of a lean mix composed of these materials produces an increase in permeability, but the effect on a rich mix is not marked.

10. In these tests the imperviousness of the concrete increased rapidly with the age of the specimens for the first month; thereafter the change was not marked.

11. From the tests thus far made it seems probable that the permeability of lean concrete in a direction normal to the pouring is greater than in the direction of pouring.

Pipe-Laying Methods on the Narrows Siphon of the Catskill Aqueduct, New York City

The general features of the Narrows siphon of the Catskill Aqueduct were described and illustrated in ENGINEERING NEWS of Apr. 30, 1914, pp. 946-948. The present article and illustrations give some additional points, developed by the construction work, which are taken from a recent description of the project prepared by the engineers of the Board of Water Supply of the City of New York.*

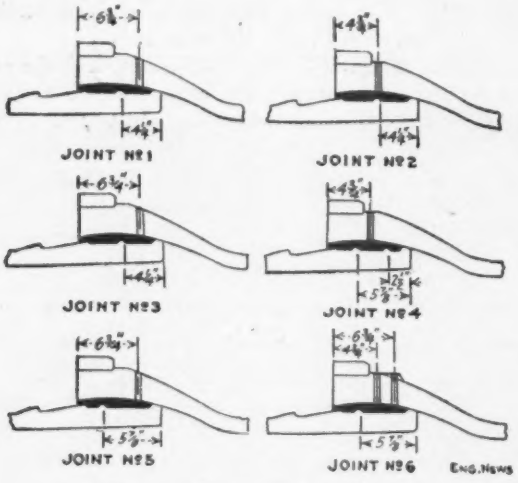

FIG. 1. EVOLUTION OF JOINT

The Narrows pipe line, which is being constructed to supply Staten Island with water from the Catskill Aqueduct, will consist of a 36-in. cast-iron flexible-jointed submarine pipe, about 10,000 ft. long, containing in all approximately 4000 tons of pipe.

Before deciding on this type of construction, many schemes were investigated. A pressure tunnel, such as has been used on the main line of the Catskill Aqueduct, was not well adapted to the conditions on account of the comparatively small quantity of water to be delivered, the length of the crossing, the great depth of the underlying rock, the difficulty of making additional borings, and the long time required for exploration and construction. The estimated cost of construction of a shield-driven tunnel was very much greater than for this pipe line.

The crossing has many of the usual difficulties encountered in laying submarine pipe, and in an exaggerated degree. The Narrows is the main entrance to New York harbor from the ocean; through its channels pass the largest steamships afloat, and very many craft of all kinds, making it one of the busiest channels in the world.

The pipe line will cross anchorage grounds for merchant vessels and ships of war. Across the anchorage ground where the depth of water is now 30 ft., dredging to about 50 ft. below the harbor bottom for a length of nearly 3000 ft. is required.

Government regulations require that no refill shall be placed above El.—45, mean low water, and protection of the pipe requires about 8 ft. of refill, so that the bottom of the pipe must be at least 56 ft. below mean low water. This great depth made it advisable to do as much of the work on the surface as possible, and therefore the continuous method of laying pipe by means of a cradle was preferred to any method of lowering the pipe in sections and joining them together on the bottom.

The modified Ward joint, known as the Duane joint, was selected as the highest development of the flexible joint to date. Experiments were made with a view of adapting it to the excessively rigorous conditions met in the Narrows. This joint is essentially a ball-and-socket joint, the socket being the bell of one pipe, and the ball the lead-covered spigot of the adjoining pipe.

On account of the shrinkage of the lead, all poured joints leaked excessively until calked at the face. The effect of the calking penetrates only ¾ in. into the lead, and as the pipe is deflected while laying at least 5°, corresponding to a horizontal movement of 2 in. along the line of the joint, the effect of any face calking done on the scow would be destroyed while laying. To obtain a tight joint, calking would therefore have to be done by divers after the pipe had reached the bottom. This would be extremely laborious and expensive on account of the great depth and strong tidal currents which limit the hours divers can work.

Experiments were conducted prior to letting the contract with a view to developing a joint which would not require calking under water. Various alloys of lead with tin and antimony were tried with a view to developing

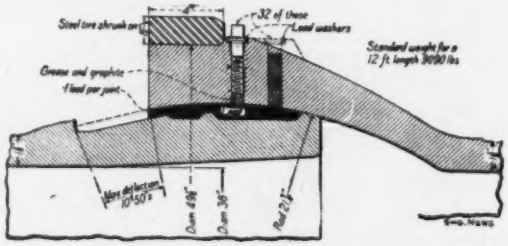

FIG. 2. FINAL FORM OF JOINT

a nonshrinking joint filler, but without success. Joints calked with lead wool proved tight but not flexible.

It was decided to attempt to calk the entire joint through mechanical means by forcing lead into the back of the joint after pouring. Sixteen holes were bored and tapped through the bell as shown on Joint 1 (Fig. 1) in the accompanying sketch. A paste made of flake graphite and anti-friction compound was put into these bolt holes and forced into the joint by screwing in tap bolts or gib screws. Lead plugs ½ in. in diameter and 1 in. long were then screwed down, about seven pellets per hole being used, amounting to nine pounds of lead for the whole joint.

The joint proved to be tight and flexible under ordinary conditions and the promise of ultimate success was sufficient to proceed with the letting of the contract. The contract was awarded Mar. 2, 1914, to the Merritt & Chapman Derrick & Wrecking Co., at an estimated total cost of nearly $1,000,000.

There were still extraordinary conditions to be met and some details to be worked out. Accordingly, through the coöperation of the contractor, experiments were resumed after the letting.

In addition to tightness and flexibility it was desired

*On the occasion of an excursion to the work by the Municipal Engineers of the City of New York and the Brooklyn Engineers' Club, Oct. 10, 1914.

FIG. 3. TYPE OF JOINT USED ON NARROWS SUBAQUEOUS PIPE-LINE

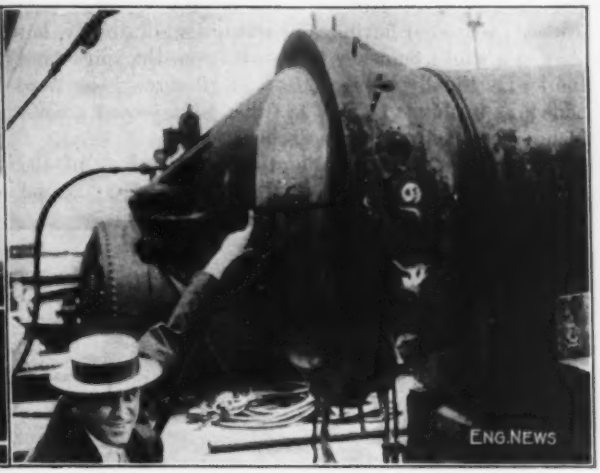

FIG. 4. BULKHEAD FOR CLOSING PIPE TO MAKE PRESSURE TESTS OF JOINTS

to get a joint which would have the greatest possible strength against longitudinal pull. The ordinary joint, even though uncalked, becomes tight when subjected to a longitudinal strain, but when the strain is released the leakage is worse than before. On account of the length of the Narrows siphon, sag could not be relied upon to tighten the joints, and in addition the stress and movements due to changes of temperature had to be considered.

It was with a view to increasing the longitudinal strength that the gib-screw hole was moved to the position shown in Joint 2 (Fig. 1), with the idea of concentrating the lead at the great diameter of the bell. This was not successful as the lead tended to flow freely out of the face of the joint instead of filling it up.

Joint No. 3 (Fig. 1) was a repetition of the preliminary experiments on Joint 1, using a larger quantity of lead in order to gain strength. Though this joint was satisfactory in many respects it still failed to give the desired longitudinal strength, because after a certain amount of lead had been forced in, further injection drove the spigot out faster than it filled up the space in the joint.

Accordingly Joint 4 (Fig. 1) was tried, which proved to be very tight and to have a great deal of longitudinal strength; but is was not flexible, for it tended to

FIG. 5. PIPE-LAYING CRADLE USED ON THE NARROWS SIPHON, CATSKILL AQUEDUCT, NEW YORK CITY

pry instead of to rotate. Joint 5 (Fig. 1) was then evolved; it proved flexible and strong longitudinally, but showed a slight tendency to leak between the spigot and the lead. Finally the second set of gib-screw holes were added, as in Joint 6 (Fig. 1); this joint proved a complete success and is the one being used.

The procedure of pipe jointing on the deck of the scow is as follows: The inside of the bell is coated

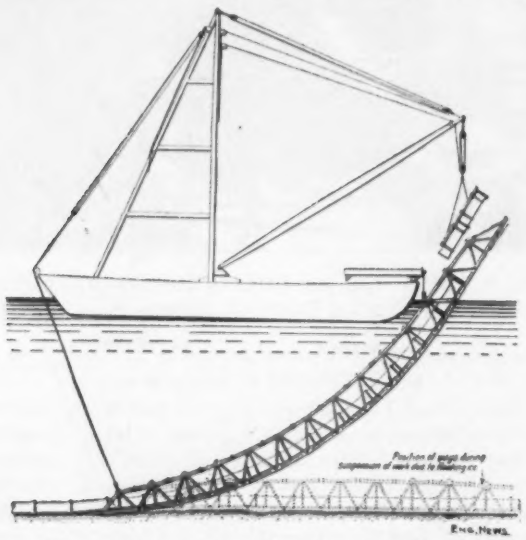

FIG. 6. DETAIL OF PIPE-LAYING CRADLE

with graphite. The spigot of the next pipe is inserted and properly centered and the joint poured, using about 300 lb. of lead. Lead pellets $\frac{9}{16}$ in. in diameter by 1¾ in. long are forced in by compressed air, three each in the back holes, three each in the front holes, and then one each in the back holes, using from 20 to 22 lb. of lead in all.

A mixture of grease with about 10% graphite is forced in each hole, and the holes are plugged with tap bolts and lead washers. The joint is deflected 5° and tested under a hydrostatic pressure of 100 lb. per sq.in., before launching from the scow, as described in the specifications (reprinted in the previous article above referred to).

The pipe-laying scow measures 40 ft. by 125 ft. It is held in position by 10 anchors. It carries the 70-ton derrick which is used for handling pipe and for auxiliary operating. At the beginning of the work it supported the pipe-laying cradle or skidway. The latter is now carried by cables running over a trunnion in the front of the scow. The cradle, or skidway, is made of structural steel and resembles a through truss, curved in the vertical plane. It is 168 ft. long, 8 ft. wide and 10 ft. high and weighs nearly 60 tons. The pipe is held in place on the skidway by three guides.

As soon as a joint has been completed and painted the skidway is drawn ahead and the finished pipe at the lower end slips off to the bottom of the dredged trench. The skidway is then ready to receive another length of pipe at the top.

The work described is being done by the Board of Water Supply, of which J. Waldo Smith is Chief Engineer; Alfred D. Flinn, Deputy Chief Engineer; Thaddeus Merriman, Department Engineer, and Thomas H. Wiggin, Senior Designing Engineer. The work is under the city department, of which Walter E. Spear is Department Engineer; John P. Hogan, Division Engineer; James F. Murphy, Section Engineer, and E. P. Hopson, Section Engineer, in charge of Narrows siphon laying. W. L. Chapman is Secretary of the Merritt-Chapman Derrick & Wrecking Co., in charge of the Narrows siphon work for the contractor, for whom Ralph Chapman is Mechanical Engineer.

Wheel Pit and Elevator for Coach-Repair Yard; Michigan Central R.R.

The convenient handling of wheels and axles is one of the essentials in the design of a coach-repair yard, and the renewal of wheels of passenger cars in the new coach yard of the Michigan Central R.R., at Detroit, Mich., is effected by means of a wheel pit and two elevators. Fig. 1 shows the deck framing of the pit, and Fig. 2 shows the elevator construction.

The pit is about 36x38 ft., and 7½ ft. deep, with a deck framing of I-beams 18 in. apart, arranged in two

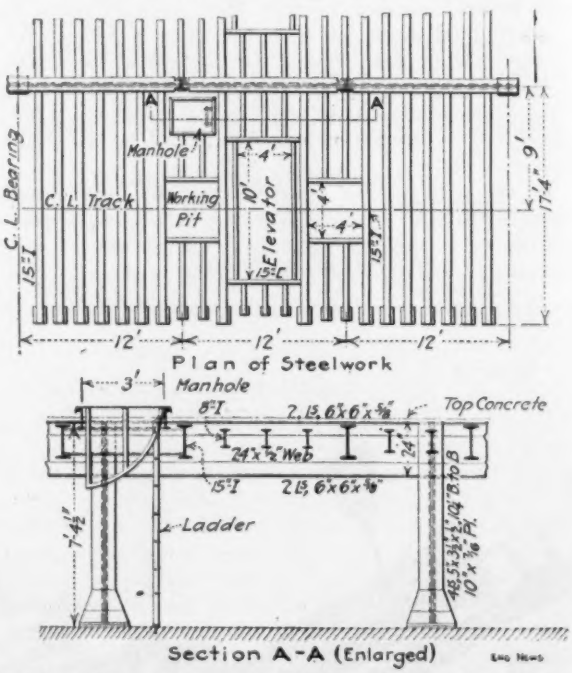

FIG. 1. PLAN OF TRACK DECK OF WHEEL PIT IN THE COACH YARD OF THE MICHIGAN CENTRAL R.R. AT DETROIT, MICH.

spans. Their outer ends rest on bearing plates on the concrete walls and their inner ends are framed against a girder supported on columns and the side walls. In each span is an elevator opening 10x4 ft., crossed by a repair track, and the elevator platform has a section of track about 4 ft. long, which forms a part of the repair track when raised and part of the wheel track in the pit when lowered. The elevators are operated by pneumatic cylinders, as shown in Fig. 2. Compressed air is taken from the pipes of the air-cleaning system in the coach yard, being supplied from compressors located in the basement of the station building.

At each side and end of the elevator opening is a pit about 4 ft. square. The floors of these pits are suspended about 4 ft. below the track and form platforms for the men working under the car, while they are high enough to allow wheels on the pit track to pass under them. These shallow working pits are covered by movable deck sections when not in use, as shown in Fig. 2. There

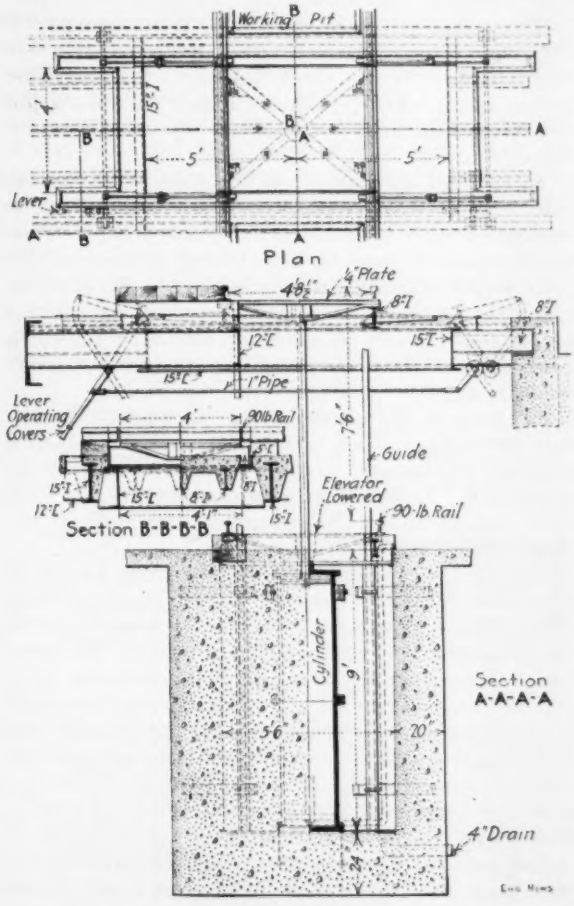

FIG. 2. SECTION OF WHEEL PIT AND ELEVATOR

is also a manhole opening about 36x30 in., having a hinged cover with segmental guides. This gives access to an iron ladder.

The operation will be understood from the plan, Fig. 3. The team-track yard has a general elevation about 4 ft. lower than the repair yard. The wheel-loading track leads out from this yard, so that the floor of a flat-car in the wheel dock is just about level with the tracks in the repair yard, enabling wheels to be loaded or unloaded directly onto the track lying across the end of the repair tracks. Thence the wheels are placed in the storage yard, or if one of the repair tracks is empty they may be rolled directly to the elevator and taken down into the wheel pit under the repair tracks. A capacity of about four pairs of wheels is provided on either side of the elevator on this low-level track. New wheels are stored on one side and the other side is reserved for the removed wheels.

Coaches on the repair track are placed with the wheels to be removed directly on the elevator, so that they can be lowered as soon as released from the boxes. The elevator takes the wheels down, and they are rolled off and the new wheels rolled on. The elevator then ascends, bringing the axles with their journals in the bearings. When the repair track is again clear, the removed wheels are brought up on the elevator and rolled away into the storage yard. The principle of this wheel pit was taken from one in use in the Mott Haven yard of the New York Central R.R. At that yard, however, the wheels are stored on the lower level, to which they are brought by

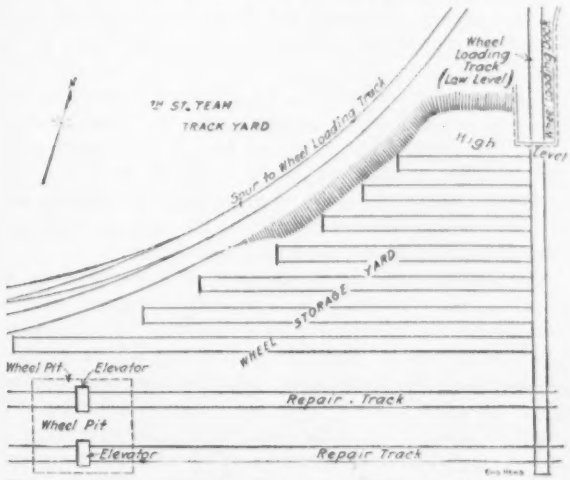

FIG. 3. PLAN OF WHEEL-STORAGE TRACKS AND COACH-REPAIR TRACKS WITH WHEEL PIT

an inclined plane, and are taken in and out from the side of the pit instead of being taken up and down on the elevators, the latter being reserved exclusively for removing and replacing the wheels under the cars.

The Export Trade in Iron and Steel and the relative prospects of Great Britain and the United States in securing the trade heretofore held by iron and steel producers on the continent of Europe are discussed by London "Engineering" of Sept. 18. The world's total export trade in iron and steel is about 20,000,000 tons per annum, of which about half has been supplied by the European continental countries now at war; 6,000,000 tons by Germany alone. Were the war to end shortly, it is assumed that financial and other losses would prevent Germany and Belgium from playing a large figure in the iron and steel export trade for some years to come. At present the financial stringency the world over has cut down the demand for export iron and steel to very small proportions. It is assumed, however, that this condition will gradually disappear and that the normal demand for iron and steel and its products from non-manufacturing countries will be reëstablished. "Engineering" concedes that in the competition for this business the United States firms will have the great advantage over the British firms of surplus capacity able to turn out the material required. It estimates the total capacity of American steel producing concerns at 30,000,000 tons of steel ingots a year, and the normal demand in the United States at 20,000,000 tons, leaving a margin of 10,000,000 tons available for export trade. Great Britain, on the other hand, has little surplus capacity, and when its home demands and the export trade it holds are normal, it can provide only a small part of the necessities of the remainder of the world. As to the relative cheapness of production in the two countries, "Engineering" thinks that Great Britain now has the advantage. In the past fifteen years the cost of iron and steel-making in the United States has materially increased, due to increases in ore royalties, cost of mining, cost of coal and coke, and falling off in the quality of ore. It is now estimated that the cost of iron and steel production in the United States has increased in the neighborhood of $6 per ton since 1899. No similar increase has taken place in England, and "Engineering" therefore believes that with the advantage the English iron and steel works have of location near tidewater, they can export at prices as low at least as their American competitors.

A Method for Constructing a Tunnel under the East River at New York City

By Emil Diebitsch*

The construction of the tube tunnels for the Pennsylvania Railroad under the East River at New York City showed the uncertainty of progress by the shield method of tunneling in this location. Sometimes the heading was lost due to a blowout and it required weeks of labor to recover it. At other times several headings had to be shut down in order to turn all available air into the headings under operation. At all times enormous volumes of air had to be pumped into the headings and finally portions of the river bed had to be covered with clay in order to make any progress at all possible.

Similar difficulties were encountered in building the present subway tunnels to Brooklyn and in both instances the cost of the work was very high.

The fundamental cause of most of these troubles is found in the difference in hydrostatic pressure at the bottom and the top of a tunnel heading. Air pressure sufficient to keep the floor of a tunnel dry will be enough greater than the hydrostatic head at the roof of the tunnel to allow the air to escape in great volume; and when the cover is not sufficiently heavy, may even blow a hole through the river bottom and by thus suddenly reducing the air pressure in the tunnel allow the water and mud to enter the tunnel, resulting in a "blow-out."

In practice this difference in pressure is kept as small as possible by keeping the air pressure in the tunnel about equal to the hydrostatic head at the middle of the tunnel section; but it is still greater than the hydrostatic pressure at the roof of the tunnel; and this difference in pressure is the chief cause of the delays and expense in building tunnels by the shield method.

In the shield method, then, the work of excavation and construction is carried forward in a vertical plane or heading under a condition of unbalanced pressures. In the method herein proposed the work is carried on in a horizontal plane under a condition of balanced pressures. For this reason this new method permits of building tunnels more expeditiously and ecomonically and incidentally also the tunnels can be built to easier grades and of better construction.

The method in general consists of building successive lengths of the tunnel under the protection of a huge, movable pneumatic caisson which rests on the river bottom and extends above the surface of the water like a dock or pier.

When a section of the tunnel has been completed, the caisson is moved forward along the line of the proposed tunnel, so that another section of tunnel immediately adjoining the finished section and connecting with it may be constructed. By repetition of this process the entire tunnel between pierhead lines may be built.

This caisson is really a huge steel box with double walls inclosing ballast chambers, surrounding a working chamber decked over on top but open at the river bottom. Water is excluded from this working chamber by means of compressed air and access to it for men and materials is gained by means of shafts and air locks in the usual way.

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The word caisson is used to designate this steel box but it should be noted that this structure differs in design and function from the caissons used for foundations of bridges and buildings. It does not form the bottom of the finished structure as ordinary caissons do but is used as a temporary protective covering placed over the portion of the tunnel under construction and is moved to a new position as soon as the tunnel section is completed.

This caisson rests on the river bottom above and at the edge of the excavation within which the tunnel is built. It therefore has no cutting edge but a broad base, giving ample bearing surface for stability and to prevent the caisson from sinking too deeply into the mud of the river bottom.

To facilitate the handling of the caisson, it is provided with ballast chambers which can be readily filled or emptied of water by means of valves and pumps. These ballast chambers must be large enough to contain sufficient weight to hold the caisson securely on the river bottom when the tunnel construction is proceeding. Enough of the ballast should be water for easy handling which, when pumped out of the chambers, will permit the caisson to float.

Reference to the accompanying drawings will show that the caisson under consideration has the following principal dimensions: Length 120 ft.; width 50 ft; depth 62 ft. Length of working chamber 100 ft.; width 40 ft.; height 25 ft. The estimated weight without machinery or ballast is 2500 tons.

This structure is large enough and strong enough to form when resting on the river bottom, a pier to which scows, lighters, tow boats, etc., may be moored and on which materials and supplies may be landed. The main deck of the caisson will be equipped with derricks, hoisting engines, concrete mixers and such other machines and appliances as may be needed for expeditiously and economically handling the excavated materials from the working chamber to the scows and the structural materials such as sand, gravel, cement, steel and lumber from the lighters to the air locks.

The machinery deck, located about 12 ft. below the main deck, will contain boilers, air compressors, steam engines, electric generators, motors, pumps and such other machinery as may be needed in constructing the tunnel or handling the caisson.

The walls of the working chamber are provided with interlocking tongued-and-grooved poling boards or shutters which resemble sheet piling. These sheet piles are shown in the drawing as constructed of steel I-beams with grooved timbers bolted on each side of the webs of the beams. These sheet piles are used as vertical poling boards and are carried down a little in advance of the general excavation and serve to keep the air within the excavation from escaping too freely. They also sustain the banks of the trench within which the tunnel is built. It should be noted that each sheet pile is attached to the wall of the working chamber by means of bent steel plates forming a groove within which the sheet pile can readily be moved vertically up or down but which resists lateral displacement of the sheet pile.

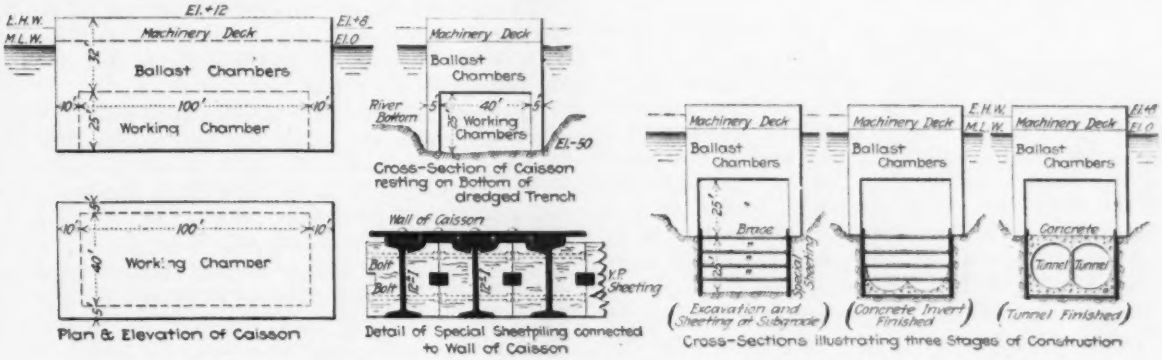
Having constructed the caisson as above described with all necessary men and material locks, ballast chambers, etc., and equipped it with air compressors, boilers, pumps, dynamos and other necessary machinery, the procedure is as follows:

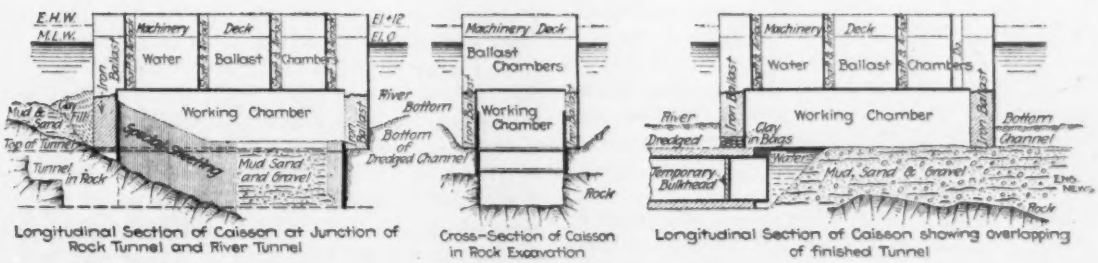
A trench is dredged along the line of the proposed tunnel, wide enough to hold the caisson when resting on the bottom of the channel and at least 45 feet below extreme low water, as the harbor regulations require the top of the tunnel to be below that depth.

The caisson is towed to a point selected for beginning work and sunk by flooding the ballast tanks until it rests on the bottom of the channel. Compressed air is forced into the working chamber until all of the water is excluded, some of the water being pumped out if deemed more expedient, and more ballast is added in order to hold the caisson in place.

Workmen now enter the working chamber and begin excavating, taking care to keep the sheeting below the excavation, and cross bracing the sheeting if necessary, as the excavation proceeds. When the bottom of the excavation is reached, concrete is placed to form the foundation and invert for the tunnel lining.

The tunnel can now be built as designed—whether of





SKETCHES ILLUSTRATING A PROPOSED CAISSON METHOD FOR CONSTRUCTING SUBAQUEOUS TUNNELS

massive or reinforced concrete with waterproofing of fabric and pitch or asphalt or brick laid in asphalt. Should it be found desirable to surround the tunnel with a metal casing to act as waterproofing, this can be made of thin steel plates or of sheet copper.

When a section of the tunnel has been completed, the sheet piling must be drawn and secured to the walls of the working chamber after which the caisson is to be moved ahead.

The next step is to float the caisson by discharging ballast and to move it along the line until the rear end just overlaps the finished portion of the tunnel, thus allowing the end of the finished section to project into the working chamber. The caisson must now be sunk to the bottom of the channel in this position by flooding the ballast chambers, after which the work now proceeds as before.

These operations would serve for constructing the portion of the tunnel under the river, but provision must be made for connecting the river tunnels with the tunnels on the Manhattan and Brooklyn shores.

A study of the boring sheet made by the engineers of the Public Service Commission for the new subway tunnels from Whitehall St., Manhattan, to Montague St., Brooklyn, for which contracts were recently let, show that approaching the Manhattan shore the rock in the river apparently rises sufficiently above the top of the tunnel to permit of driving a tunnel in the rock in the ordinary way. In this case the caisson could be located near the point where the tunnel enters the rock from the sand and mud in the river and a heading could be started into the rock from the working chamber of the caisson. If the rock is of such a character as to require compressed air to keep the excavation dry, or compressed air and a shield, the air could be furnished from the plant on the caisson and the tunnel driven toward the shore, using the caisson as a shaft.

If on the other hand the rock is solid and the tunnel can be driven without air, it might be advisable to bulkhead the tunnel after it has been well advanced into the rock and drive the rest of the tunnel under normal air pressure from a shaft on the Manhattan shore.

On the Brooklyn shore, where the tunnel enters earth and sand below mean high water, it may be advisable to use a shield. The shield could be set up in the working chamber of the caisson and operated with the plant on the caisson, using the caisson as a shaft for men and materials.

In considering the operations involved in this method of constructing tunnels, the following questions suggest themselves:

(1) The construction of a caisson of the magnitude indicated and the character of the stresses to be taken care of.

(2) The facility of handling a caisson of the dimen-

sions indicated, floating it, moving it into line, sinking it to the bottom of the river and subsequently raising it again. In this connection the force of the currents in the river must be considered.

(3) The stresses in the special sheathing which is used to carry the excavation below the bottom of caisson. The possibility of constructing this sheathing so that it will stand the strain and also be practically air-tight.

(4) Leakage of air from working chamber and trench.

(5) The advantages which this method of tunnel construction has under the existing conditions over the method of using compressed air and shields.

Construction of Caisson

In considering the first question, we will assume the following dimensions of the caisson: Length over all 120 ft., width 50 ft., depth from top to bottom 62 ft., length of working chamber, 100 ft., width 40 ft., height above bottom of caisson 25 ft.

When such a caisson is resting on the bottom of the river in 50 ft. of water, and a trench 25 ft. deep has been excavated below the edge of the caisson, the air pressure in the working chamber and trench necessary to keep water out of the bottom of the trench must be equal theoretically to the hydrostatic head of 75 ft. which is the distance from the surface of the water to the bottom of the trench. This hydrostatic head of 75 ft. is equal to a pressure of 32.55 lb. per sq.in. or 4687 lb. per sq.ft. This pressure produces a very heavy load on the walls and roof of the working chamber.

The stresses will not be discussed in detail here but they have been sufficiently developed to show that they can easily be taken care of with a steel structure weighing 2500 tons.

The working chamber is made 25 ft. high so that the sheet piles can be made in one piece and drawn up clear of the bottom of caisson when the latter is to be moved. The height of the working chamber and the fact that no cross braces are shown in the chamber increases the stresses in the walls but these increased stresses have been considered and the 2500-ton structure will take care of them.

The high working chamber offers several advantages in the economical handling of excavated materials, braces, steel lining, concrete, etc., which do not, however, affect the principle of the method of construction and hence will not be discussed in detail here. These economies more than offset the cost of the additional steel due to absence of braces in working chamber.

Handling Caisson

Considering now the facility of handling this caisson in the water, we will assume that the caisson weighs 2500 tons and that the machinery placed upon it will weigh 500 tons. The caisson is 120 ft. long by 50 ft. wide and therefore displaces 6000 cu.ft. of water for every foot that it sinks into the water. Six thousand cu.ft. of water weighs 375,000 lb. or 187.5 tons. Since the caisson without ballast weighs 3000 tons it will sink 16 ft. into the water and by adding ballast can be sunk to any desired depth.

As it is proposed to rest this caisson on the bottom of a trench in 50 ft. of water, and from time to time raise it a few feet, move it and sink it to the bottom again, it will be advisable to ballast with some heavy material such as pig iron so that the caisson will float at a depth of 45 ft. and can be sunk lower by adding water ballast. This will require 45x187.5 tons or 8437.5 tons less 3000 or 5437.5 tons of pig iron.

The space available for ballast is equal to the volume of the caisson below the machinery deck less the space occupied by the working chamber. The machinery deck is to be 12 ft. below the top deck and the volume of the caisson below that deck is equal to $120 \times 50 \times 50 = 300{,}000$ cu.ft. From this deduct the volume of the working chamber $100 \times 40 \times 25 = 100{,}000$ cu.ft. The space available for ballast is therefore 300,000 less 100,000 or 200,000 cu.ft. less the space occupied by structural members, shafts, etc.

Assuming that pig iron stacked as ballast will weigh 300 lb. per cu.ft., one ton will occupy 6.67 cu.ft. and 5437.5 tons will occupy 36,268 cu.ft. This leaves 200,000 less 36,268 or 163,732 cu.ft. of space for other ballast, when the caisson is floating at a depth of 45 ft.

This other ballast is required to sink the caisson to the bottom in fifty feet of water and hold it there, and as it has to be easily handled in order to sink or raise the caisson readily it will consist of water.

The maximum amount of ballast will be required when the trench under the caisson has been excavated to the subgrade of the tunnel, for there it will be necessary to have a pressure of air in the working chamber equal to a hydrostatic head of 75 ft.; this is 32.55 lb. per sq.in. or 4687 lb. per sq.ft. Under these conditions the uplift due to air will equal area of working chamber 100×40 or 4000 sq.ft. times 4687 lb. or 18,748,000 lb. or 9374 tons. To this must be added the uplift due to pressure of water on bearing area of caisson, the bottom being 50 ft. below the surface of the water and the bearing surface having an area of 2000 sq.ft.; the pressure per sq.ft. at depth of 50 ft. is 3125 lb.; total pressure equals 2000×3125 or 6,250,000 lb. or 3125 tons.

The total uplifting pressure is therefore 9374 tons plus 3125 or 12,499 tons. Against this we have weight of caisson, machinery and pig-iron ballast a total of 8437.5 tons leaving 4061.5 tons to be taken care of by water ballast.

Ballasted as above and with 32.55 lb. of air in the working chamber, the caisson will theoretically touch the bottom but not bear upon it. As it may be desirable to have the caisson press firmly on the bottom under these conditions, it will be necessary to consider the total space available for additional water ballast.

It has been shown that the space available for ballast is about 200,000 cu.ft. The pig iron occupies 36,268 cu.ft. and 4061.5 tons of water will occupy 129,968 cu.ft. Total space occupied by iron and water ballast is 162,236 cu.ft., which amount deducted from 200,000 leaves 37,764 cu.ft. of space to be occupied by structural members, shafts, etc., and additional water ballast.

From these figures it appears that sufficient water can be admitted to ballast chambers to hold caisson firmly on the river bottom under the heaviest air pressure needed for the conduct of the work.

In order to sink and raise the caisson readily, the ballast chambers will be provided with valves for the ready admission of water and with pumps for the discharge of ballast water with reasonable dispatch.

It has been shown that the maximum amount of ballast will be in the caisson where the trench has been ex-

cavated to subgrade and there is 32.55 lb. per sq.ft. air pressure in the working chamber. Under these conditions it has been shown that there will be at least 4061.5 tons of water equal to 162,236 cu.ft. or 1,216,770 gal. in the ballast chambers. Four 6-in. centrifugal pumps, each with a capacity of 3000 gal. per min., will empty all of this water in 102 min. But with all of the water out of the ballast chambers the caisson will float at a depth of 45 ft. and as it is only necessary to clear the caisson of the trench bottom in 50 ft. of water, it will theoretically be necessary to pump out only about 80% the above quantity of water for the caisson will float in 50 ft. of water with 937.5 tons or 300,000 cu.ft. or 225,000 gal. of water in the ballast chambers. From these calculations it can be readily seen that the ballast chambers can be emptied entirely in less than 1 hr. 42 min. and sufficient for manipulating the caisson in 1 hr. 23 min.

The flooding either ballast chambers or working space, it will only be necessary to furnish a sufficient number of flooding valves of proper size to fill these spaces in a very short time.

The space to be occupied by water ballast is to be divided and subdivided by longitudinal and transverse bulkheads so as to form separate ballast chambers. These chambers are to be so connected with the pumps and so interconnected with each other that any ballast chamber or any group of chambers can be emptied or flooded independent of all others. This arrangement facilitates very much the handling of the caisson.

The operation of sinking and floating a structure similar to the caisson under consideration is neither novel nor untried as is evidenced by the ease and rapidity with which the floating gates of the dry-docks in our Navy Yards are manipulated in docking and undocking vessels. The great floating dry-docks at New Orleans, Manila and elsewhere are operated in exactly this principle and battleships weighing ten thousand tons or more are lifted out of the water in a few hours.

There is, however, one condition in connection with this tunnel construction that is different from those existing in the operation of floating dry-docks and that is the possible interference caused by currents.

Inspection of the tide tables published by the U. S. Coast & Geodetic Survey shows that the currents at the site of the proposed tunnels run in opposite directions at different stages of the tide and that every six hours there is a period of slack water when no currents exist.

The maximum velocity is given at 3.1 miles per hour and the pressure exerted by such a current against a caisson 120 ft. long and 50 ft. deep is 127.6 tons. Tendency to displace the caisson is resisted by the pressure of the caisson on river bottom and by the sheathing driven below bottom of caisson as well as by the spuds which will be operated from the main deck.

The end of the caisson which rests over the finished portion of the tunnel is made about 4 ft. less in depth than the forward end and the long side walls. This shortening of the rear wall prevents the weight of the caisson from bearing too heavily on the unyielding masonry of the finished tunnel while the other portions of the caisson bottom bear more or less on the compressible river bottom.

This notch in the rear end of the caisson also permits the caisson to ride a finished portion of the tunnel when the caisson is being moved forward. The finished tunnel will thus serve as a guide for the caisson and also resist the action of the currents toward moving the caisson laterally.

The space which occurs between the bottom of rear wall of caisson and the roof of the finished tunnel where it projects under the rear end of the caisson into the working chamber after the caisson has been settled in place for a new tunnel section can be filled with bags of clay carefully packed to prevent air leakage and further protected by lowering the sheet piling on the rear wall until it rests on the tunnel roof.

Operations for moving and sinking caisson would be conducted as nearly as possible during periods of slack water when the currents are of low velocity or nonexistent.

The difficulty of moving caisson and resetting it in a current of the velocity of 3.1 miles per hour is evidently not very great for in constructing the Detroit River tunnels, sections of the lining of two tunnels 23 ft. in diameter and 260 ft. long were sunk in two hours in a current of about 3.4 ft. per sec. or 2.3 miles per hour.

In considering the remote possibility of the caisson escaping from its moorings and being carried away by the currents, it should be remembered that the caisson draws 45 ft. of water even when all the water ballast has been pumped out, consequently it cannot float beyond the confines of the dredged trench as it would run aground.

Sheathing

In considering the sheathing which is used in carrying the excavation 25 ft. below the bottom of the caisson, it should be borne in mind that the plan of procedure contemplates driving the sheet piling in advance of the excavation to a greater or less extent. The distance driven will be determined by the character of material penetrated, the absence or presence of boulders and the ease of penetration.

It will be necessary to increase the air pressure in the working chamber so as to exclude the water from the bottom of the excavation as the trench is deepened. As a result of this increase in air pressure there will be a pressure on the sheeting tending to force it against the material in the river bed back of the sheeting.

The maximum pressure will be exerted when the bottom of the trench has been reached 75 ft. below the surface of the water, for then the air pressure in the trench and working chamber will be 32.55 lb. per sq.in. or 4687.5 lb. per sq.ft., sufficient to resist the hydrostatic head of 75 ft. which exists outside of sheeting at the level of the trench bottom.

Considering the stresses in the sheet piling under these conditions of maximum air pressure, it is found that on a level with the bottom of the excavation the air pressure in the trench tending to push the sheeting against the banks is just equal to the 75 ft. of hydrostatic pressure which exists outside of the sheeting and tends to push the sheeting into the trench. These two equal and opposing forces counteract each other and there is no strain on the sheeting at the bottom of the trench.

As the air pressure within the trench and working chamber is uniform throughout, while the pressure against the outside of the sheeting decreases as the surface of the water is approached, the resultant pressure is from the trench outward and increases as the upper end of the sheeting is approached.

At the upper end of the sheeting, on a level with the bottom of the caisson 50 ft. below the surface of the water there is a hydrostatic head of fifty feet against the outside of the sheeting while within the trench at the same level the air pressure is equal to the hydrostatic head of 75 ft. The resultant pressure equal to a hydrostatic head of 25 ft., is the equivalent of 1562.5 lb. per sq.ft.

A sheet pile 12 in. wide and extending from the bottom of the caisson downward 25 ft. to the bottom of the trench would therefore be subjected to a pressure varying from nothing at the bottom to 1562.5 lb. per sq.ft. at the top. The tendency to force a sheet pile under such pressure outward is resisted by the material in the river bottom outside of the sheeting, and as ordinary soils have a safe bearing capacity of 1½ tons per sq.ft., even the maximum pressure of 1562.5 lb. at the top of the sheeting is not sufficient to push it out of place.

As a matter of fact, the wall of sheet piling separating the air pressure within the trench from the hydrostatic pressure on the outside merely acts as a diaphragm to distribute the comparatively small inequalities of pressure between the two opposing forces.

In designing this sheeting it should be made stiff enough and strong enough to take with safety the maximum pressure of 1562 lb. per sq.ft. for a span of from six to ten feet to take care of soft spots, blow-holes, cavities made by dredging which may exist in the banks of mud sand and gravel against which the sheeting will bear.

Furthermore, when the caisson is located over a rock bottom it may be advisable to drive sheeting to rock, make a tight connection with the rock, brace the sheeting across the trench and then reduce the air pressure in the trench to normal. In such a case the pressure will be from without in and the maximum pressure would be at the bottom of the sheeting. The sheeting should be sufficiently strong to resist this pressure for a span of about ten feet between waling pieces and braces.

To make a practically air-tight sheeting a tongued-and-grooved sheeting of yellow pine, such as is ordinarily used in deep trench work, suggests itself. As such a wooden sheet pile might not be strong enough for all conditions, a strong and tight sheet pile can be made by bolting yellow-pine timbers to each side of a steel I-beam and using the composite beam as a sheet pile. The wooden members are to be tongued and grooved in the usual manner, thus insuring a sliding joint with the adjoining sheet piles which can be maintained practically air-tight.

Each sheet pile will be attached to a wall of the working chamber by means of a groove built of steel in which the sheet pile can readily slide up or down. This sliding connection will be so constructed that the sheeting cannot be displaced laterally in any direction without rupturing the steel plates of which the groove is to be built.

Sheeting constructed in this manner will be prevented from drifting when driven into the river bottom, thus insuring tightness and will also require less waling and bracing as it is carried down.

Air Leakage

There will be practically no excessive air leakage at the bottom of the excavation, for here the inner and outer pressures are equal, but when the excavation has been carried down to subgrade there is a difference of about 11 lb. per sq.in. between the pressure within the work chamber and on the outside at the level of the bottom of the caisson.

Under these conditions the air tends to escape between the back of the sheeting and the walls of the caisson and thence under the bottom of the caisson and through the water into the free air. To reduce this leakage it will be advisable before starting to drive the sheeting to pack bags of clay under the inner edge of caisson and fill all cavities which may exist between caisson bottom and river bottom with clay. These bags of clay are to be so placed that the sheet piles, as they descend, will press firmly against the clay and make a close contact.

After the sheet piles have been carried to the full depth, the space between sheet pile and caisson wall can be calked with oakum or other suitable materials or the space may be "mudded up" with clay in the usual way.

Advantages of This Method of Construction

From the foregoing description and discussion it is evident that this method of tunnel construction offers the following advantages over the shield method.

1 Safety from Blowouts—The balanced pressures in the "horizontal heading" or plane of operations removes all danger from blowouts and flooding of working chamber and thus assures (*a*) the safety of the workmen, (*b*) steady progress in the work of construction.

2 Easier Grades—As no cover other than the caisson is required the tunnels need not dip as far below the river bottom as shields necessitate and hence easier grades can be followed.

3 Exact Line and Grade—The tunnel may be built to exact line and grade, results which are approximated in the shield method by making the diameter of the tube larger and correcting deflections from line and grade in the inner concrete lining of the tube.

4 Foundations—The river bottom may be reinforced with piles or by other methods should conditions demand it, before the tunnel is built.

5 Better Construction—As the tunnel is accessible from all sides the work of construction is open to thorough inspection and the tunnel may be built of riveted steel plates surrounded with impervious concrete. This gives a tighter, less flexible and more permanent structure than one built of cast-iron plates bolted together and placed directly against the river mud. The expensive operation of pumping grout blindly into the mud in the hope that eventually it will fill all cavities back of the tunnel lining, is also obviated.

6 Materials Handled More Economically and Expeditiously—The caisson resting firmly on the river bottom forms a stable dock of ample area, equipped with derricks and buckets for the handling of excavated materials direct to scows and lighters and for the receiving of materials of construction such as sand, gravel, cement, steel, lumber, etc., from lighters and barges moored alongside the caisson.

By this arrangement the cost of conveying excavated material through the finished portion of the tunnel to the shaft, hoisting it at the shaft, dumping it into wagons, carting it through the streets and dumping it on barges at the dock is obviated. Similar operations in bringing materials of construction to the heading are dispensed with.

7 Work Done More Economically and Expeditiously—It is evident that with an area 100 ft. long and 40 ft. wide uncovered for work and protected from interruption from "blowouts" more men can be employed and the work finished more quickly and economically than by working through shield doors against a tunnel face 20 ft. in diameter.

There is also less labor involved in hoisting or lowering materials through shafts directly overhead than there is in conveying them through the finished tunnel to a shaft on shore.

8 Generally More Economical—In addition to handling materials and labor more economically there is a considerable saving in the amount of plant and in the operation of the same, in the amount of piping, cost of building shafts, cost of pushing shields through rock tunnels on the Manhattan side of the river from the shaft on shore to where the rock dips below the tunnel line in the river deposit, cost of property where plants are located, cost of covering part of river bed with clay and cost of pumping great quantities of compressed air which escapes through the river bed. The balanced pressures in the heading, the tightness and comparatively small volume of space under air pressure reduce the requirements for air to a minimum and hence reduce the number of compressor units required.

The location of the plant on the caisson directly above the "heading" reduces the amount of piping required, while the location of shafts and air locks directly over the operations does away with the necessity of tracks and cars used in the shield method to convey materials from the heading to the shaft on shore and *vice versa*.

The setting up of shields in the working chamber of the caisson as the river tunnel approaches the shore and the subsequent use of the caisson as a shaft for handling materials does away with the necessity of sinking a shaft on shore. This is a great saving as the cost of such a shaft is large.

As the caisson covers the section of the tunnel under construction there is no occasion to cover the river bottom with a blanket of clay for there is no excessive air leakage.

Estimated Cost

An estimate carefully made in detail shows the following costs exclusive of contractors' profit:

Caisson with machinery complete.....	$400,000
Operating caisson	118 per day
Interest charges	66 per day
Supervision, office and field.....	183 per day
Tow boat for men.....	75 per day

With the above overhead charges and the actual labor and materials estimated at present market prices, a tunnel 18 ft. in diameter made of ⅜-in. steel plates and surrounded by 3 ft. of concrete would cost if constructed by this "caisson" method $245 per lin.ft. through mud, sand and river deposit and $370 per lin.ft. through rock.

The estimated cost of such a tunnel built on the location shown for the new subway tunnels from Whitehall St., Manhattan, to Montague St. Brooklyn, would be $3,176,330. If to this cost 25% be added for contingencies and the profit of the contractor, a reasonable contract price for the tunnel would be $3,975,412 or practically $4,000,000.

The lowest bid received by the Public Service Commission for the construction of a tunnel to be built on the same route, but according to the plans and specifications of the Public Service Commission, was $6,368,550 if built alone, and $5,974,809 if built in conjunction with the tunnel from Old Slip Manhattan to Clarke St.,

The Explosibility of Grain Dust

A number of disastrous grain-dust explosions in elevators and mills have occurred during the past few years and have attracted general attention from engineers. An arrangement was made between the U. S. Bureau of Mines and representatives of various milling interests for a scientific study of the explosibility of grain dust and of methods to prevent such explosions. Work was started late in 1913 under direction of G. A. Hulett, Chief Chemist of the Bureau of Mines, with D. S. Price in charge of field investigations and H. H. Brown in charge of laboratory work. A preliminary report has been issued from which the following data are abstracted.

An investigation was made of 13 explosions which have occurred since 1905; of these, three were in Iowa, three in New York, two in Illinois and one each in Vermont, Indiana, Kansas, Ohio and Texas. The explosions were distributed among various lines of milling as follows: cereal mills 4, elevators 3, feed mills 2, starch factories 2, glucose factory 1, flour mill 1. In these 13 explosions 78 men were killed and 119 injured; the total property damage was in excess of $2,000,000. The relative freedom of flour mills from explosions compared with the magnitude of the industry is noteworthy; this has been attributed by millers to the elimination of the old dust bin, which took place after the disastrous explosions in the great flour mills of Minneapolis a quarter century ago.

Various wheat, corn and oat dusts from different parts of mills and elevators were analyzed and their ignition temperatures and explosion pressures were determined. As the basis of comparison, Pittsburgh standard coal dust was used in accordance with the Bureau of Mines practice.

In general, it was found that the grain dusts are more inflammable than coal dust, higher explosion pressures being developed in most cases and especially at the lower temperatures (between a range of 900° to 1300° C). Dusts from oats and yellow corn appeared to be more inflammable than dust from wheat. The explosion pressures with dried dusts were some 16 times greater than with the undried.

The investigators found several causes generally assigned to the explosions noted: open-flame lights, conflagrations, foreign materials in grinding machines, sparks from electric motors, switches and circuits, and static electric discharges from belts. It was found that many of the mill explosions and fires originated from foreign materials in the grinding machines and it is recommended that some means be found of removing such materials before the grain reaches the mills. It is also recommended that only a completely guarded electric lighting system be used, that portable electric lanterns be used in place of flame types, and that receiving bins from grinders be kept as small as practicable to decrease the space for dust clouds.

Granite-Block Pavement as Smooth as Asphalt

A granite-block pavement as smooth as one of sheet asphalt seems an anomaly, yet engineer visitors to Worcester, Mass., may satisfy themselves of the truth of this statement: that the granite-block pavements of Main and Front Streets, which have been down ten or more years, are as smooth as ordinary sheet-asphalt pavements which have been down two or three years. Fig. 1 is from a photograph of Main St. just after a street-sprinkling car had passed, so that the wet surface reflected the light and brings out all the imperfections.

A NEW YORK ENGINEER'S OPINION—Worcester, Mass., was probably the first city to make use of the cement-grout joint. The spaces between the blocks were filled with stone screenings and then poured full of the grout. The result was very satisfactory.

There are advantages and disadvantages in a joint of this kind. It makes a smooth, even pavement, and one that is easily cleaned. It presents little resistance to traffic, and brings the wear of traffic vertically on top of the blocks, so that the pavement is very durable, but becomes smooth and slippery under wear.

It is extremely necessary that the grout should become thoroughly hard before any traffic is allowed on the pavement, otherwise the blocks will be loosened and the permanent set prevented, so that the filling becomes little, if any, better than sand.

Pavements thus laid become practically continuous, giving off a metallic noise under traffic. The hardened grout, being a good conductor of sound, transmits the noise readily from block to block. Where the grade is flat, however, and the street is kept free from traffic until the grout has thoroughly set, the pavement will be very satisfactory. It is the smoothest form of stone pavement that has been laid in the United States.—Geo. W. Tillson "Transactions." Am. Soc. C. E., LXXV, pp. 527-29.

FIG. 1.—MAIN ST. GRANITE BLOCK PAVEMENT; WORCESTER, MASS.

These extraordinarily smooth granite-block pavements are due to several reasons: (1) a fine-grained, uniform granite, which is not so hard as that ordinarily specified or used; (2) experience and skill of the municipal laborers, acquired by many years' continuous experience in laying stone-block pavement; (3) a rich portland-cement grout and stone screenings for joint filling, and the method and care in application; (4) a fairly heavy traffic, which wears off the original roughness and irregularity.

THE BLOCKS

A digest of the Worcester specifications follows:

SPECIFICATIONS AND CONDITIONS—The stone blocks are of granite of a durable, sound and uniform quality, each measuring on its face or upper surface not less than 7 nor more than 12 in. in length, and not less than 3¾ in. nor more than 4¼ in. in width and not less than 6 nor more than 6½ in. in depth for the larger size blocks, and not less than 4 nor more than 4⅛ in. in depth for the smaller size blocks.

The blocks used on each piece of work are uniform in depth and are of the same quality as to hardness, color and grain. All edges are sharp and straight, and at approximately right angles with each other, and dressed so as to form, when laid, close end and side joints, with fair and true surfaces on top, bottom and ends, and are to be in all respects equal to the specimen blocks furnished by the contractor.

All blocks delivered must be satisfactory to the Street Commissioner, and any block which does not in his opinion conform strictly to these specifications may be either rejected or caused to conform to the requirements of the contract at the expense of the contractor. The blocks are delivered as ordered, upon cars, at such times and in such quantities as the Street Commissioner may direct, and no liability attaches to the city until their acceptance is made in writing.

THE PAVEMENT

The pavements are laid by the Street Department with municipal day labor, according to specifications of the City Engineer, a digest of which follows:

FOUNDATION—The foundation is 4, 5 or 6 in. of 1:3:6 portland-cement concrete.

CUSHION—Upon the concrete foundation is a layer of clean, sharp, screened, dry sand, not less than 1½ in. thick, evenly spread to the finished crown. If the sand is not dry it must be made so by artificial heat before being spread. The sand cushion is prepared at least 25 ft. ahead of the paving.

LAYING THE BLOCKS—The blocks are laid with their longest dimension perpendicular to the center line of the street, except at street intersections where the blocks are laid diagonally. Each course of blocks is of uniform width and so laid that all longitudinal joints are broken by a lap of at least 2 in. [No joint width is specified, but no attempt is made to get blocks giving much closer than ¾-in. joints.] None but competent pavers are employed. The blocks are always laid with the widest face down, and each and every one is thoroughly rammed to a firm bedding. Low or high stones are picked out and reset. Ramming is not allowed within 20 ft. of the block laying.

FILLING JOINTS—As soon as 300 ft. of pavement has been laid, the joints are filled to a depth of 2 in. with trap-rock screenings, immediately after which the pavement is rerammed, if necessary, until the surface is made solid and secure. After this ramming the joints are filled flush with trap-rock screenings, and a rich portland-cement grout.

The grout is poured upon the pavement until all the joints are full and flush, and broomed to an even surface. The grouting is repeated until no settlement of the liquid in the joints is apparent, and the grout filling remains flush with the top of the blocks. A thin layer of trap-rock screening is spread and broomed over the entire surface until the pavement is thoroughly coated with grout.

The grout is composed of 1 part portland cement and 1 part clean, sharp sand, free from moisture. The materials are mixed to a satisfactory consistency and kept constantly stirred during the operation of pouring and spreading. The blocks are thoroughly wet by sprinkling immediately before

FIG. 2. CLOSE VIEW OF FRONT ST. GRANITE BLOCK

applying the grout. After completion the pavement is kept damp and is protected from traffic until the grout has thoroughly set and hardened. [This is one of the most important provisions, for it insures that grout is given a chance to stick to the surface of the blocks.]

COST DATA—These pavements cost, complete with 1:3:6 portland-cement concrete foundation, from $2.75 to $3.25 per sq.yd. The blocks cost about $54 per 1000 for 4-in. size and $60 for 5-in.

CONCLUSIONS—THE SECRET OF SMOOTH GRANITE-BLOCK PAVEMENTS—Fig. 2 is a close view of the Front Street pavement. Although the blocks are a bluish-white granite (Fig. 3) when placed, they wear a slaty color. The joints wear with the same uniformity as the granite itself so that the surface is smooth and free from depressions. The pavement is not slippery like sheet asphalt, but has a gritty feeling.

The pavement when first placed is comparatively rough, but the grout is broomed into the surface depressions and granite bumps wear smooth; so that, if not disturbed for subsurface construction, the pavement is actually in much better condition after ten years' use than when put down; and absolutely nothing has been spent for maintenance.

The secret is not so much the method of construction, perhaps, as in the character of the granite itself. The granite used in Worcester and Lowell, Mass., where pavements of the same remarkable smoothness have been attained, all comes from New Hampshire quarries of the Hildreth Granite Co., which are within a radius of 25 mi., including the towns of Concord, Marlborough and Milford. While varying much in color and appearance, these granites all have the same general physical characteristics.

Typical of all is the so called "Concord granite" described as follows:

Concord granite is a muscovite-biotite granite of medium bluish-gray color. The texture is fine to medium, somewhat porphyritic, with sparse, slender, isolated feldspars up to 0.5 in. Although the micas occur in very minute particles, especially the biotite, they measure up to 0.2 in. and exceptionally 0.4 in.

Its constituents in descending order of abundance are: Slightly bluish, translucent potash feldspar inclosing particles of quartz and of soda-lime feldspar; clear to pale amethystine quartz with hair-line crystals, probably of rutile, and with cavities in sheets which in some sections are parallel; milk white striated soda-lime feldspar, more or less kaolinized and centrally micaized; with white potash mica (muscovite); and black mica (biotite), some of it chloritized, generally in smaller scales than the muscovite. In some cases the mica plates have their flat sides parallel to the rift direction; in some the biotite appears to predominate over the muscovite, possibly owing to the alignment of its scales.

The accessory minerals are: magnetite, apatite, zircon and rutile. Purple and white fluorite occurs here and there on and near joint planes. Calcite and quartz are associated with it on these joints. Secondary minerals are: kaolin, muscovite in veinlets, a white mica without potash, chlorite and calcite.

FIG. 3. CHARACTER OF BLOCK USED IN WORCESTER

The specific gravity is 2.64-.65 or 165.6 lb. per cu.ft. The polish is fair, but the abundant mica plates and size of some of them do not favor the durability of the polish under long-continued outdoor exposure.*

These granites contain a relatively large quantity of finely divided mica. The quarry men believe that to this is due the tenacity with which the grout clings to these blocks, the lean grout following the mica filaments into the interior of the blocks. These and the soda-lime felspars, more or less kaolinized, must make the porosity relatively high for granite. The crushing strength is from 18,000 to 23,000 lb. per sq.in. The fact that as a polished stone this granite weathers considerably is probably proof of its porosity, and possible lack of durability as a building stone; but neverthless tends to make it an ideal paving material.

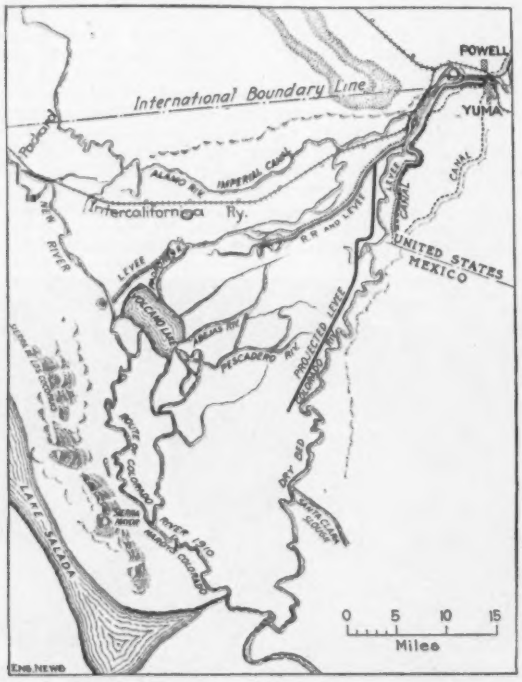
The city engineer of Worcester is F. A. McClure, to whom we are indebted for much of this information.

*Dale, U. S. Geological Survey Bulletin No. 354.

A Suggested Plan for Developing the Lower Colorado River*

By Walter D. Smith†

As irrigation engineer, United States Reclamation Service, and the first to be stationed on the lower Colorado River, the writer has had very exceptional opportunities to study the hydrography of that river. From 1902 to 1906 he had charge of the hydrographic investigations on the lower Colorado made by the Reclamation Service and the Geological Survey, including observation and official reports of all that occurred at the Imperial Canal headings in the United States and in Mexico, the diversion of the river to the Salton Sink, and the work done in returning it to its old channel.



Map of the Lower Colorado Delta and Imperial Valley, California-Mexico

(Reproduced from "Engineering News," Jan. 26, 1911 and Dec. 7, 1911.)

In 1903 he was a member of the first party to make a trip of reconnoissance into Mexico for the Reclamation Service. Further study and a trip over this region in June, 1913, has confirmed him in his opinion as to the best methods of controlling the floods.

Importance of the Project—At a recent meeting of the Engineers and Architects' Association of Southern California, A. P. Davis, Chief Engineer of the United States Reclamation Service, expressed the opinion that the Colorado River Development is a bigger project than the Panama Canal. Mr. Davis has been connected with both projects.

There is enough water, if properly stored, to irrigate more than 3,000,000 acres, and that amount of land is available. Estimating the cost of putting the water on land at $50 per acre, and the value of land with water at $100 per acre, the net profit would be $150,000,000. Estimating the value at $500, the direct net profit would be $1,350,000,000. With the opening of the Panama Canal and the flood of commerce which will pour through, there will be on the Pacific Coast a plentiful supply of labor for construction work and an enormous demand for land. Our products will have access to the markets of the world through the harbors at Los Angeles and San Diego; also perhaps over perfect railway grades through a harbor on the Gulf of California. The Colorado River country cannot be developed too rapidly.

Rapid development, however, may be attended with organic weakness. Rapid as has been the development up to the present time, it could have been more rapid and more secure if there had been a central organization with a single comprehensive plan. An illustration of this is found in the disaster of 1905. Today the situation is more complicated than it was in 1905 and there is more at stake. There should be some way of uniting the efforts of all who are concerned—private corporations, mutual water companies, reclamation service projects, War Department, individual landowners in Mexico and in five states of this country.

Fortunately in 1905 the only cost was retarding development of the Imperial Valley about two years and $3,000,000 in tangible damage and expense. It might be much worse another time. Today the Colorado River floods are not being watched and guarded against as carefully as they should be. Everybody's business is nobody's business.

A Plan to Fit the Case—The writer suggests the organization of a permanent commission composed of five members, to be known as the International Colorado River Commission, to have full responsibility and authority subject to the approval of the legislative bodies of the two countries, as follows:

1. To adopt a general plan of development to use the natural resources regardless of international boundary lines.

2. Apportion cost of same, decide method of financing and order of construction.

3. Determine by whom the work shall be done; what portion by the United States and by Mexico; what portion by private corporations, mutual companies, Reclamation Service, War Department, and what if any directly by the Commission; by whom controlled and operated when completed.

4. To review and report annually on the progress of the work and to have direct charge of all completed headings, sluiceways, reservoirs and dams on the river itself.

5. To adjust rights, national, state and individual.

Each commissioner to serve five years subject to recall at any time, the term of one member to expire each year. Two to be chosen by a water users' association composed of all irrigators using Colorado River water; one by landowners in the districts where Colorado River water is in prospect, the exact districts to be determined at first by the surveys of the Reclamation Service; one to be appointed by the President of the United States; and one by the President of Mexico.

By such a plan all interests will have due representation and all vested rights, individual and national, will be protected. The landowners who have most at stake and who will pay much of the cost will have sufficient

*An extensive article on the ineffectual attempt of the Federal Government to control the lower Colorado River was published in "Engineering News," Dec. 7, 1911, p. 667.

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direct authority to get results promptly and efficiently. In this way the two countries can agree upon progressive methods of development and land ownership.

ENGINEERING PROBLEMS—The engineering problems to be solved in the accomplishment of this work are:

(1) Store flood waters; (2) separate water from silt; (3) put water on land for irrigation; (4) keep flood water off the land; (5) drain irrigated land and use water again; (6) dispose of surplus flood water; (7) dispose of surplus silt; (8) develop electric power; (9) perhaps build land with silt.

The remainder of this article will deal primarily with points 6 and 7. The other points have already been considered by others and can be accomplished by proved methods. The most urgent problem now is flood control. It is even more important to protect and insure the territory already developed than to develop new territory. Insurance is worth while. Land in the Imperial Valley sells for $150 per acre which in the San Bernardino, San Gabriel and Los Angeles valleys would bring $500 per acre. Also in most of the territory yet to be developed the flood problem is the most important, the most difficult and the most liable to be neglected. It probably will be impossible to control the floods completely by storage reservoirs alone.

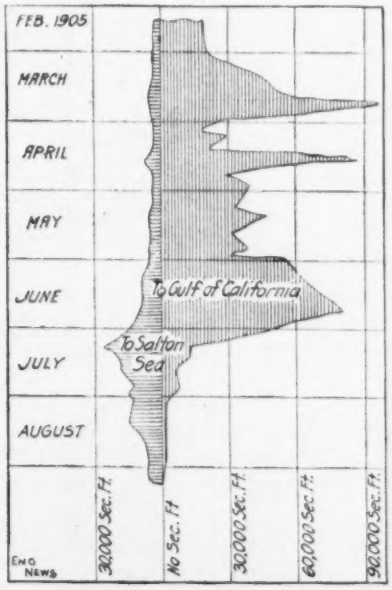

DIAGRAM SHOWING RATE OF DIVERSION OF COLORADO BY IMPERIAL CANAL, 1905

(Amount of water diverted to the Imperial Valley and Salton Sink from February to June, 1905, was only 10 to 13%; from July 1 to 15 it increased to 70%.)

To keep the river out of the Salton Basin by a levee along the high land north of Bee River and Volcano Lake is not entirely satisfactory. It will be expensive and there is danger of the river breaking through. Also the water cannot reach the ocean without using a very large tract of land as a settling basin.

To force the river into its old channel, the shortest route to the ocean, gives a better grade but no satisfactory settling area. With its load of silt amounting each year to 75 sq.mi. one foot deep, this will be increasingly expensive and insecure. If the silt could be removed this expense and danger could be eliminated.

WHERE TO PUT THE SILT—The writer suggests that the proper place to put the troublesome part of the silt is in the Salton Basin. In this way the Imperial Valley, the delta in Mexico, and the Yuma Valley can be insured against flood as they never can be by levees alone. As an additional insurance device the controlling gates might be made large enough to take a considerable portion of the flow at flood, using them as a safety valve.

When the system of storage reservoirs on the headwaters is completed, the Salton Sea may be sufficient to take all the remaining floods, and the channels to the ocean can be used as canals.

With all the power of the federal government the lands along the Mississippi have never been insured against flood, perhaps never will be by levees alone, and only at great expense otherwise. The same would be true of the Colorado if there were no Salton Sink.

The possible incidental advantages of this plan are:

1. Drainage for the Imperial Valley, of all land on north slope of the delta, and lands of the Yuma Project.

2. Eliminating expense of removing silt from canals. Sink where surplus drainage from the Imperial Valley can be used for irrigation.

7. Straightening and making permanent the Alamo Channel below Sharp's or Holt Heading.

8. Shifting of Alamo above Sharp's Heading to a permanent location closer to the mesa, increasing area of delta land easily available for irrigation.

9. Increased opportunities for development of electric power.

The details of this plan are simple. The cost will be moderate. The plan is to sluice the silt into the Alamo Channel by controlling gates on bed rock at Pilot Knob. With the heavier silt removed the old channel below Pilot Knob will scour deeper and straighter. All that is necessary is to scour deep enough to carry the flood waters without levees. Some bank revetment may be required. At 200 ft. below sea level, the elevation which the Salton Sea attained in 1907, the evaporation amounts to 2,000,000 acre-ft. per year. This amount minus the drainage from the Imperial Valley would be available for the work of sluicing. The larger the gates at Pilot Knob, the less the total amount of water required for that purpose, because: (First) the greater the discharge the greater the percentage of silt possible to carry; (second) the more the gates can be depended upon as a regulating safety device at crests of floods, the less the amount of silt necessary to remove.

CONCLUSION—The piecemeal methods which necessarily have been used up to this time have proved expensive and dangerous. The development of the Colorado River country is a case where prompt and comprehensive work is more economical than slow and partial development. With the knowledge and experience gained in the past such action is now possible. Construction can go forward rapidly and surely as soon as a practical permanent organization is effected.

Sewage Disposal by a Public Service Company is not entirely out of date yet, it appears from recent press reports from Collingswood, N. J. The Collingswood Sewerage Co. is petitioning the New Jersey State Public Utilities Commission for permission to increase its rates from $9 to an average of $20 per house. The sewerage company alleges that it has lost $92,000 in its operations of a sewerage system and disposal plant. Oct. 20, has been set for a final hearing on the case. Collingwood is in Camden County and has a population of about 5000.

Brooklyn Sewage-Experiment Plant

By George T. Hammond*

SYNOPSIS—A notable addition to the list of municipal experiment plants for the treatment of sewage has been made by the Borough of Brooklyn. This plant includes units for screening, sedimentation in Dortmund and Imhoff tanks, a special sludge-digestion tank and Imhoff sludge beds, a sewage-aërating tank and sewage-aërating siphon, sprinkling filters, both ordinary and extra-aërated by means of compressed air, and final sedimentation tanks. Disinfection studies are proposed, as also tests of a Reinsch-Wurl screen and a Moore vacuum filter. The sewage flow to all units is measured by calibrated orifices in connection with constant-level devices and the compressed-air supply for aëration is measured by Venturi meters.

A unique series of working-scale experiments in sewage treatment is being carried out by the Bureau of Sewers of the Borough of Brooklyn, New York City. The experimental plant is located at the 26th Ward pumping station and sewage-treatment plant, on Jamaica Bay. The studies there being made are intended to assist in designing new sewage works for the 26th Ward, in place of the present outgrown plant, and also to assist in designing works for other portions of Brooklyn. Part of the plant was put in operation on Oct. 4, 1913, and most of the rest on Jan. 1, 1914.

At the 26th Ward site and also at other available sites for disposal works for Brooklyn the problem is to provide works which will produce an effluent suitable to be discharged into tidewater, which works must be located on marsh land with a surface only slightly above sea level.

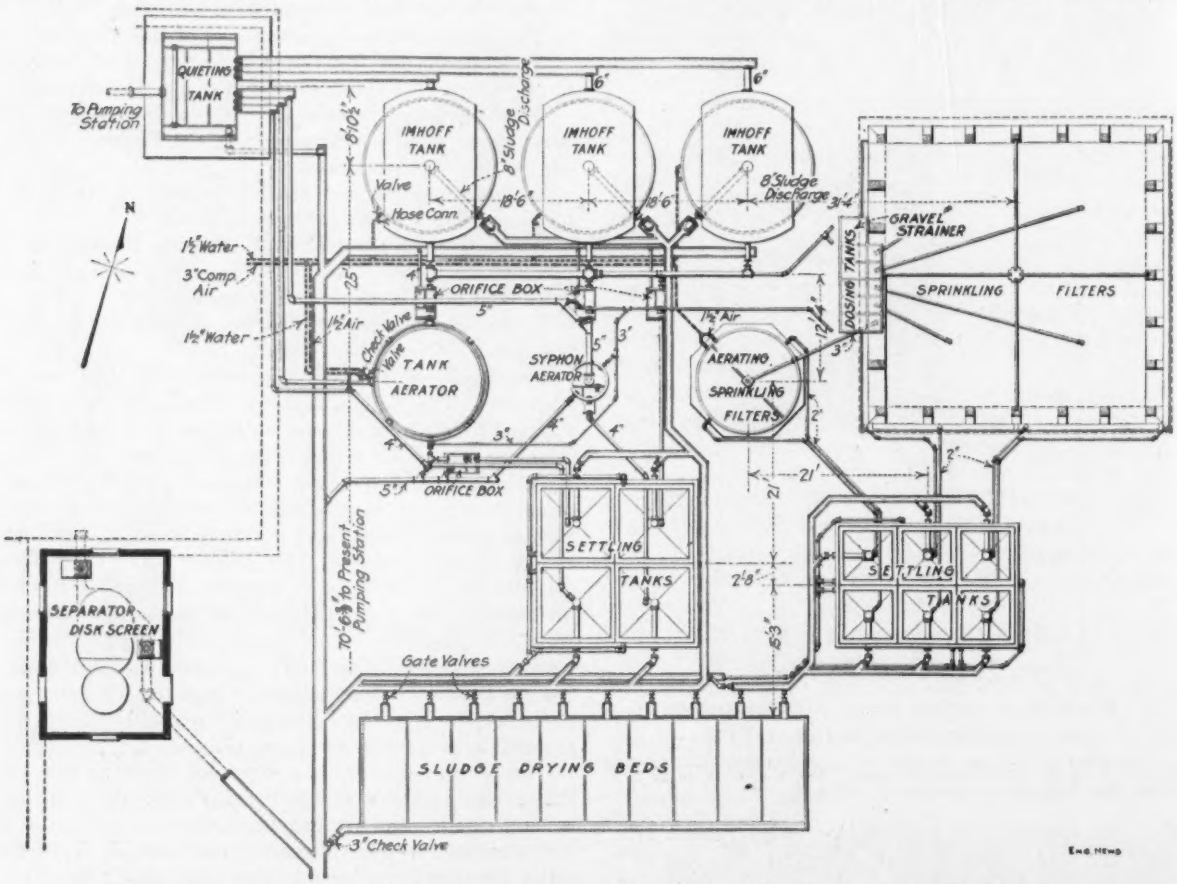

Fig. 1. General Plan of Experimental Sewage-Purification Plant, Brooklyn Borough, New York City

The probable degree of purification required has so wide a range as to make it desirable to consider various methods of screening, sedimentation, sludge disposal, oxidation, and combinations of these, as well as disinfection. It was also considered desirable to add to the knowledge of the hydraulics of sewage and sludge flow in sewage-treatment plants and the effect of very cold weather on exposed sprinkling filters. The opportunity to study the effect of shape and structure on the operation of the various experimental units was also seized.

For the foregoing and other reasons all parts of the experimental plant were placed above tidewater and made

*Designing Engineer, Bureau of Sewers, 215 Montague St., Brooklyn, N. Y. This article was prepared for "Engineering News" several months ago. Subsequently Mr. Hammond was asked to submit a paper on the same subject to the American Society of Municipal Improvements, on Oct. 6. Out of courtesy to the Society our article, which necessarily traverses the same ground as the paper, but which is briefer, was deferred until after the presentation of the paper.—Ed.

accessible for observations on sewage and sludge and on the structures themselves. For study there were selected a Reinsch-Wurl screen and a Moore vacuum filter (or strainer); a Dortmund tank for plain sedimentation,

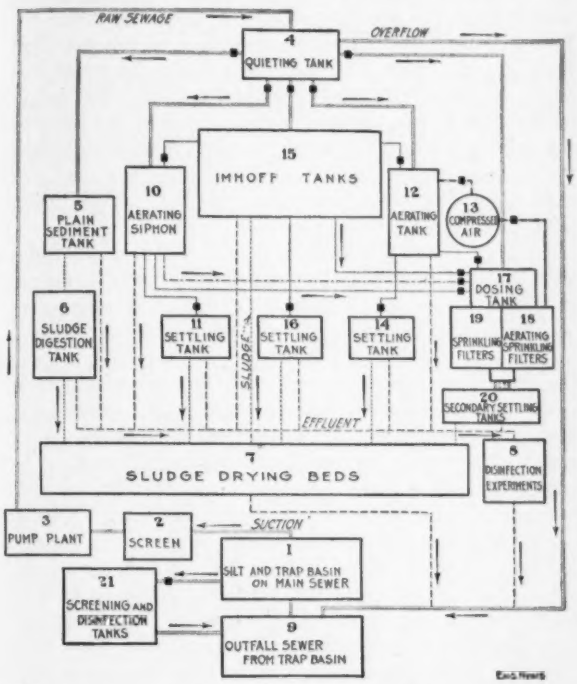

FIG. 2. DIAGRAM SHOWING RELATION OF APPARATUS IN VARIOUS STAGES OF OPERATION AND EFFECT ON SEWAGE, BROOKLYN SEWAGE-EXPERIMENT STATION

coupled with an independent sludge-digestion tank, and also three Imhoff tanks of different depths, coupled with sludge beds (the latter also being available for sludge from other units); an aërating siphon and an aërating tank, each supplied with compressed air, ordinary sprinkling filters and sprinkling filters supplied with compressed air; secondary Dortmund settling tanks receiving effluent from the aërating siphon, aërating tank and Imhoff tanks; and disinfecting apparatus. To aid in the hydraulic studies and in keeping accurate quantity records there were provided a knife-edge weir 25.84 ft. in length (ENG. NEWS, Apr. 9, 1914), in the outfall sewer; a quieting tank to receive the sewage from the pumps and distribute it by gravity to the various primary elements of the experimental plant; various orifice boxes and calibrated orifices to measure the flow of sewage to each unit; and Venturi meters to determine the volume of air used for aëration.

Fig. 1 shows the actual layout of the plant. The sequential relations of the various parts and processes are shown by the diagram, Fig. 2. A general view of the plant is shown by Fig. 3.

The large district from which the sewage used in the experiments comes is sewered on the combined plan. The sewage reaches the 26th Ward station in two 12½-ft. twin sewers, which terminate in a silt chamber. The latter is continued, in turn, by an outfall flume to tidewater. A 48-in. bypass conveys the dry-weather flow from the silt chamber to the old treatment works, from which it is pumped back into the flume.

The sewage for the experimental plant is drawn through a 12-in. suction pipe to a 1,200,000- and a 650,000-gal. direct-acting steam pump, and is discharged through an 8-in. pipe 180 ft. long into the quieting tank already mentioned. The maximum lift is 40 ft. After supplying the quieting tank for some time with unscreened sewage it is proposed to try, successively, screens of different mesh on the 12-in. suction pipe. The Reinsch-Wurl screen will be tested with raw sewage, which has not gone to the quieting tank. The air for aëration purposes is supplied by a duplex air compressor, automatic in stopping, starting and speed. The compressor has a rated displacement capacity of 228 cu.

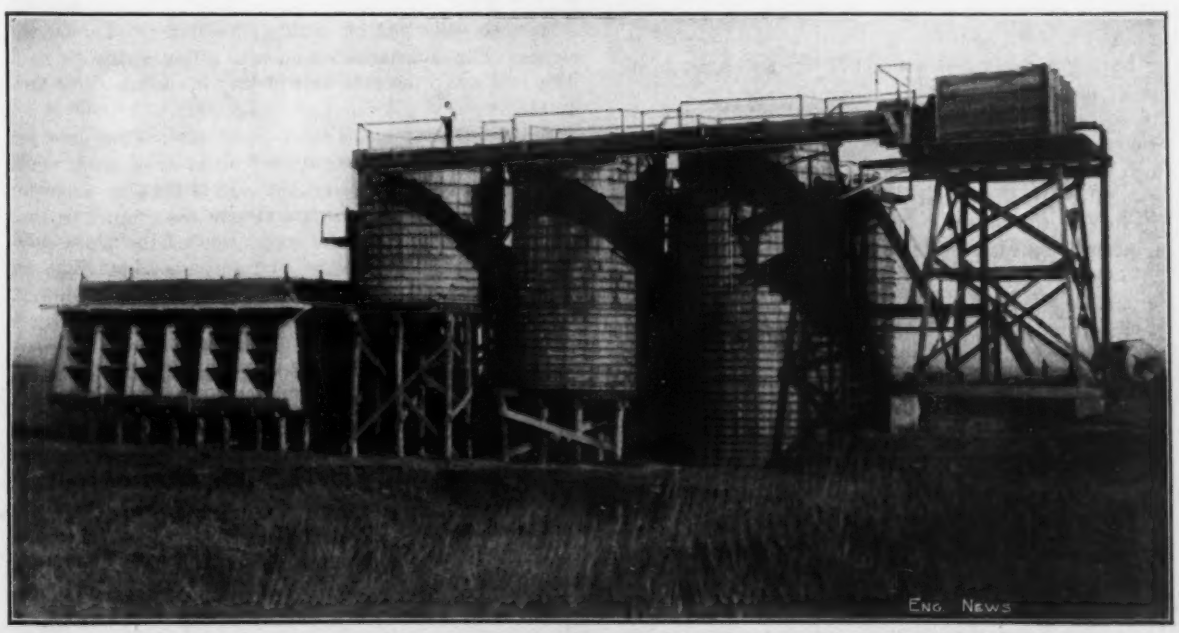

FIG. 3. GENERAL VIEW OF BROOKLYN SEWAGE-EXPERIMENT PLANT

ft. of air per min. and delivers into a receiver 24 in. in diameter and 72 in. high.

The maximum elevation of the land at the site of the experimental plant is 1.5 ft. above high water. The sludge beds were made the governing elevation of the experimental plant, with their surfaces at 2.67 ft. above mean high water. The elevation of the quieting tank was fixed so as to give a gravity flow throughout the plant. For this purpose the constant level of the sewage was set at 33.42 ft. above datum, or 32.92 ft. above the invert of the main outlet of the sludge-bed underdrainage system.

In general, the structural elements of the plant are of wood, but concrete was used for the Dortmund settling tanks, and for the ordinary sprinkling filters, and steel for the special sludge-digestion tank.

Quieting Tank—By referring to Figs. 1 to 3, it will be seen that the center of operations is the quieting tank, which occupies the commanding elevation already noted. This tank is 5 ft. deep and 12 ft. 3 in. by 8 ft. 10 in. in plan. In its interior, each end is divided off by a partition, forming overflow chambers, which are connected with the waste pipe; the main chamber is between these internal partitions, and is 9 ft. in length by 8 ft. in width. Each end partition is cut down to the water line a horizontal distance of 3 ft. from the outlet side of the tank, forming an overflow wier, designed to maintain a constant hydraulic head over the outlet orifices. Two baffles are placed lengthwise of the tank, normal to the line of flow. The baffle nearest to the entrance of the sewage extends 3 ft. upward from the bottom, and is 2 ft. 6 in. from the entrance side of the tank. The other baffle extends downward from the top of the tank to within 1 ft. of the bottom, and is 2 ft. 6 in. from the first baffle, parallel with it, toward the outlet side. Thus the sewage, which enters submerged, rises over the first baffle, and then passes downward under the second, before entering the constant-level part of the chamber, from which it is fed to the various units through adjustable orifices.

Orifices—In the side of the main chamber of the tank, opposite to the entrance of the sewage, are placed six outlet orifices, each provided with an adjustable opening having a calibrated scale, which may be set to any rate of discharge within its range of capacity.* These orifices discharge directly into flumes that lead to the various experimental units, each orifice into a flume which it serves. The orifice boxes installed throughout the plant, with their weirs, are designed upon the same principle as the quieting tank. They were included in the general contract for the experimental plant.

A separate contract was made (with the Wallace & Tiernan Co., Inc., of 136 Liberty St., New York City) for supplying, installing and calibrating in place, the adjustable-orifice fixtures and also the compressed-air meters on the supply lines to the forced-aëration devices. The specified capacity of the orifices ranged from 150,000 to 600,000 gal. per 24 hours. As already stated, orifices are used to control the flow to each unit of the plant.

Sewage Clarification and Sludge Disposal

A considerable portion of the plant was devoted to sewage clarification and to sludge reduction and disposal. For these purposes both Dortmund and Imhoff tanks were used, as well as a separate sludge-digestion tank and sludge-drying beds.

Dortmund Tanks—Of a group of four Dortmund tanks, one is used for primary sedimentation and three for secondary sedimentation, the latter receiving the effluents from the aërating siphon, aërating tank and the Imhoff tanks. Each Dortmund tank is of concrete, 8x8 ft. in interior plan, and 8 ft. deep from the water line to the lowest point in the pyramidal bottom. The inflow is through a 4-in. central down-pipe and the outflow through V-notches, two on each side. The sludge from the primary tanks goes to the sludge-digestion tank and that from the secondary tanks to the sludge-drying beds, in all cases without pumping.

Imhoff Tanks—The three Imhoff tanks, which differ only in depth and cubical capacity, supply settled sewage or effluent to seven experimental units, all of which may be in action at the same time: (1) The ordinary sprinkling filters; (2) aërating tank; (3) aërating siphon; (4) the special sprinkling filters, with compressed-air aëration; (5) a secondary settling tank; (6) disinfection tanks; (7) Moore mechanical vacuum filter tanks.* Sewage discharged to the various sprinkling filters may be passed through a gravel roughing filter. Sludge for testing and drying goes to the drying beds and surplus sludge is discharged into the sewer.

The only parts of the Imhoff tanks which are of the same depth in all three tanks are the scum or baffling boards, which dip 18 in. into the sewage. Later on, different baffling arrangements will be studied. Each tank is 15 ft. in diameter, with depths from the water line downward as follows:

	Total or center depth, ft.	Sedimentation-chamber depths, ft.	
		Vertical side	Center
Tank 1	30.38	9.22	13.97
Tank 2	21.88	5.30	10.05
Tank 3	13.67	2.42	7.00

The floors of the upper stories or sedimentation chambers have a slope of about 42°. The floors do not lap, or pass, as is usually the case, but below them there is a timber shield, with upper slopes of 45°. In ten months' service no collection of settlings has remained upon the slopes. The tanks are covered with a flat removable roof. The tanks have been in satisfactory operation since Oct. 4, 1913.

Sludge-Digestion Tank—This tank (the location not shown on the accompanying plan) is of steel, made to be water-tight and air-tight. It is 5 ft. in diameter and 15 ft. in depth, set vertically in the ground so that no part of it is above the water line of the plain sedimentation tank, from which it receives sludge. The effect of varying temperature or of chilling from ground water is minimized by providing a double shell, with a closed air space between the shells. The sludge is drawn from the primary Dortmund tank through an inspection box to the digestion tank. The digested sludge is passed to the sludge-drying beds.

Sludge-Drying Beds—There are 10 beds, of the Imhoff type, each 5x12 ft. in plan. The medium is 8 in. of "steam" ashes, surfaced by 1 in. of coarse sand, the whole supported on a timber platform.

Aëration Devices

Either raw sewage from the quieting-tank or Imhoff-tank effluent may be subjected to aëration in (1) an aërat-

*See more detailed description, with illustrations, in the Field and Office Section of this issue.—Ed.

*The last three not yet in use.

ing siphon, (2) an aërating tank, (3) sprinkling filters equipped with a compressed-air grid, or (4) ordinary sprinkling filters. The two first named aërating devices will here be briefly described, and the third and fourth will be taken up under Sprinkling Filters.

Aërating Siphon—This was adapted from a design used on a working scale, but under unfavorable conditions, at Millville and at Riverside, N. J. It is an application to sewage of the siphon air compressor, proposed by the late C. C. Beddoes, who obtained a patent covering the use of the siphon for sewage aëration, and the process. It may be operated with raw sewage or Imhoff effluent.

The flow of sewage is led by gravity to the bell, or enlargement at the top of the siphon pipe, into which the sewage falls, sucking the air in with it, and passing vertically downward with considerable velocity to such a depth that the entrained air becomes compressed, and the sewage exposed to it absorbs a greater proportion of air in consequence of the pressure; as the volume of air absorbed will be in proportion to the head or height of the volume above it.

The apparatus consists of a 4-in. pipe, extending vertically downward 130 ft., first through the center of a vertical tank, specially designed for this experiment, 30 ft. deep and 4 ft. in internal diameter, which rests on piles. The remaining 100 ft. of the 4-in. pipe passes downward inside of an 8-in. pipe, from the bottom of the 4-ft. diameter tank to below the bottom of the 4-in. pipe, which it inclosed. This leaves an annular space, through which the sewage returns from the bottom of the 4-in. pipe to the vertical tank, in which the sewage is retained for a period of time in proportion to the quantity of flow and capacity of the tank. The sewage leaves near the top of the vertical tank by means of a waste pipe and measuring-orifice box, from which a portion of the flow is piped for observation to one of the four secondary settling tanks.

Aërating Tank—This tank was developed from experiments made at the 26th Ward sewage works in 1911 by Col. Wm. M. Black and Prof. Earle B. Phelps. The tank is 12 ft. in diameter and 25 ft. 8 in. in height. The sewage enters at the top of the tank by gravity from the quieting tank, or the Imhoff tank, as the case may be, through a pipe surrounding the top. The sewage may fill the tank so that the points of entrance are submerged, or the tank may be operated at lesser depths of content. The tank effluent is taken off at the bottom.

A grid for supplying compressed air is placed on the bottom of the tank, 7½ in. of broken stone passing a 2-in. ring and retained by a 1-in. ring being provided to support it, and the same depth of broken stone being placed over it. The grid consists of two 1½-in. pipes at right angles, forming a cross connected in the center, the arms of the cross being connected with quarter circles of ¾-in. pipe forming concentric rings, of which there are five. Each ring is perforated at 6-in. intervals with $\frac{1}{16}$-in. holes. The air enters through the 1½-in. pipes and is distributed thereby to the rings and is discharged into the broken stone surrounding the grid, which tends to break up any upward streaming effect. The main outlets for the tank effluent are 1 ft. above this grid. Through the central axis of the tank is placed a vertical 4-in. pipe, which serves to center and support the deflector disks, of which there are nine, placed longitudinally, provided for the purpose of deflecting the downward flow of sewage and upward flow of air bubbles, so as to obtain even distribution of both air and sewage.

The deflectors are designed in the form of a wheel with a hub, which is of iron. Six radial arms are provided, between which slats are placed. The slats are set in grooves in the arms at an angle of about 45° with the horizontal; the slats in each alternate deflector being set at angles alternating from and toward the center.

Ordinary and Extra-Aërated Sprinkling Filters

Much of the interest in the plant centers in two sets of sprinkling or percolating filters, one set of four beds being of the ordinary type and the other set of two beds being equipped with a compressed-air aërating device, so arranged that one bed may be tested with and one without special aëration at the same time. Before the sewage reaches the filters, it may be passed through a gravel roughing filter and must pass through dosing tanks. The filters may be supplied with: (1) Crude sewage from the quieting tank; (2) aërated sewage from the siphon aërator; (3) aërated sewage from the aërating tank; (4) Imhoff-tank effluent; (5) any of the foregoing after having been passed through the gravel-roughing filter; (6) sewage passed through the Reinsch-Wurl screens. All the sewage is applied by gravity, except that from the screens, which will have to be pumped. The screens referred to are not yet installed.

Gravel Roughing Filter—This is a 12-in. bed of gravel passing a ¾-in. ring and retained on a ½-in. ring, supported on a wire screen of ¼-in. mesh.

Dosing Tanks—There is a dosing tank for each filter, equipped with calibrated orifices and constant-head overflow. Each dosing tank is also equipped with counters that record the number of discharges. The dosing tanks discharge into pyramidal feeding tanks.

The elevation of the water line in the dosing tanks at the instant of siphon discharge is 26.74 ft., which is 9.40 ft. above the surface of the filter beds. The elevation of the same line in the reversed pyramidal feeding tanks is 23.5 ft., which is 6 ft. 2 in. above the beds.

Ordinary Sprinkling Filters—There are four of these, square in plan, and each 0.005 acre in effective area, separated by 4-in. wooden partition walls and inclosed outwardly by reinforced-concrete piers, cast with slots for receiving 3-in. wood shutters sloping inward at an angle of 45°. The piers carry a reinforced-concrete coping, level with the top of the filter bed. The beds have concrete floors, on which rest 6-in. half-tile underdrains, convex side up. The filter medium consists of 10 ft. of broken trap rock. The size of stone in each bed is different, as follows: No. 1, passed a 1½-in. ring and retained on a ¾-in. ring; No. 2, 2 to 1 in.; No. 3, 2½ to 1¼ in.; No. 4, 2½ to 1¾ in. Provision is made for taking samples at depths of 6, 7¼, 8½ and 10 ft.

Extra-Aërated Sprinkling Filters—A tank 12 ft. in diameter and 16 ft. high is divided vertically into two equal parts, alike in every respect, with independent 6-in. half-tile underdrainage systems, resting on concrete. The beds are filled to a depth of 10 ft. with 2½ to 1¼-in. stone. This depth may be increased to 14 ft. if desired. Each bed is provided with a grid near its bottom, formed of ¾-in. iron pipe, perforated every 6 in. with ⅛-in. holes, through which metered compressed air is supplied. As already stated, one bed is operated with, and the other without, extra aëration, for comparative purposes.

SPRINKLER NOZZLES—Various types of these will be tried; the initial installation is of the Taylor type. A board fence around and between each bed of each set serves as a wind shield to prevent unequal distribution.

SECONDARY SETTLING TANKS—The secondary settling tanks provided for these filters are six in number, placed in a group, each tank being a reversed truncated pyramid 10 ft. deep from the water line. The flow enters through a 2-in. pipe carried down to a point 2 ft. above the bottom. A 6-in. sludge pipe acts under the hydraulic head of the tank. The tank effluent is taken off by means of troughs passing entirely around the top of each tank, two V-shaped openings being provided on each side of each tank. The effluent passes to the main sewer. The sludge removed may be studied upon drying beds.

The experimental plant was designed and constructed and is operated by the Bureau of Sewers, Brooklyn Borough, New York City. E. J. Fort is Chief Engineer of the Bureau of Sewers.

The Salem Water-Works and the Salem Fire*

BY FRANK A. MCINNES† AND CLARENCE GOLDSMITH‡

The point of origin of the Salem fire (June 25, 1914) in Boston St., between Pope and Proctor streets, was immediately adjacent to a 20-in. main having a normal pressure of a little more than 50 lb. The building in which the fire started, and those surrounding it, were of frame construction and mutually exposed each other, thus enabling the fire to spread with such rapidity that the adjoining buildings were almost immediately involved. Two of these were protected by automatic sprinklers. Before the first hour had lapsed, 10 engines were at work and a conservative estimate based on a careful study of lengths of hose lines, sizes of nozzles, and water pressures maintained at the engines, shows that at no time during this period was more than 2500 gal. of water per min. pumped from the distribution system, yet the recording pressure gage maintained by the Water Department on Church St. registered a drop in pressure from its normal of 42 lb. to 20 lb. at the end of the hour.

At the beginning of the fire the 20,000,000-gal. reservoir was practically full, the two 5,000,000-gal. pumping engines were in operation under a full head of steam, the 30-in. and 20-in. supply mains were in service. No breaks occurred in the street mains during the progress of the fire.

A test of the carrying capacity of the distribution system was made on Aug. 31, 1914, as follows:

Four chucks were set on hydrants (near the point of origin of the fire). These hydrants were opened, two discharging through three 2½-in. outlets and two through four 2½-in. outlets. In addition, a post hydrant in an adjacent alley was discharging through two 2½-in. outlets. The flow from each of the 16 outlets was measured by Pitot tubes and showed a total of 8585 gal. per min. The drop in pressure in the 20-in. main at the point where the test was made was observed on the pressure gage attached to a hydrant which was not discharging. The gage showed a pressure of 54 lb. prior to opening the hydrants and 22 lb. while they were discharging, giving a drop of 32 lb. The recording gage on Church St. showed a pressure of 47 lb. before the test and a pressure of 27 lb. while the hydrants were discharging. The quantity of water obtained shows an abundant supply at this point, an amount in excess of the combined capacity of the engines working at the end of the first hour.

A draft from the mains equal to the previously estimated quantity pumped by the engines, namely, 2500 gal. per min., would cause a local drop in pressure of less than 4 lb. On the date of the fire, the recording gage on Church St. showed 42 lb. pressure when the first alarm was received. In 30 minutes the pressure had dropped to 33 lb., and at the end of the hour to 20 lb.

From this record it is evident that a quantity of water approximately equal to the quantity obtained at the flow test was being withdrawn from the system and the principal object of the investigation here recorded was to account for this large rate of flow. One 8-in. and two 6-in. connections supplying the two sprinkler risks already referred to must bear the blame. The 8-in. connection from the 20-in. main was through 25 ft. of pipe to the sprinkler equipment in the Charles H. Keefe plant, The two 6-in. connections supplying the sprinkler equipment in the Carr Leather Co.'s plant was fed through 200 ft. of 8-in. main connected to the 20-in. pipe. These buildings ignited immediately and a large number of sprinkler heads opened in a very short time but were powerless to check the fire. The collapse of the buildings resulted in the breaking of the sprinkler pipes, allowing the capacity of the three large connections to be wasted from the 20-in. main. To determine the actual effect of these three broken pipes they were opened shortly after the flow test above referred to was made, with the following result:

A gage on a hydrant on the 20-in. main showed a normal pressure of 54 lb., and when the connections were open and flowing, a pressure of 31 lb., which, from the previous flow test, enabled the discharge from these pipes to be accurately determined at 7200 gal. per min.; a quantity of water in excess of the entire supply required for the fire protection of a city the size of Salem according to the best modern practice. The Church St. gage fell during this test from 46 to 33 lb. The effect of these broken connections is further evidenced by the record of this gage which continued to drop during the fire until it registered 13 lb. about 3 p.m., at which time a supply at the rate of 8,000,000 gal. per day was delivered from the Peabody system without improving the pressure; at about 5 o'clock the heat abated so that these connections, together with a post hydrant, which had been discharging through two 2½-in. outlets, were closed and the pressure immediately rose to a little over 21 lb. in spite of the fact that many service pipes were broken at this time.

The lesson to be learned from the experience in Salem is that the existence of connections from a distribution system of such size that their failure destroys the pressure at the hydrant is a mistake that surely invites disaster to a water-works systems and doubly so one depending upon hydrant hose streams.

The record of fires occurring in sprinkler risks during the past 16 years shows that in a total of 14,353 fires

*Abstract of a paper read before the New England Water Works Association, Boston, Mass., September, 1914.

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‡Engineer High Pressure Water Service, 53 Bristol St., Boston, Mass.

13,691 were controlled by the opening of less than 51 sprinklers, and only 662, or 4.6%, were not controlled by this number of sprinklers, which number can be supplied by a 4-in. connection.

In most closely built-up sections the first piece of apparatus will be on the ground within three minutes of the alarm of fire and additional water may be immediately delivered to the sprinklers through outside siamese connections with which all sprinkler equipments should be provided. The fire department would then have the supply under its control and be able to deliver the water at the seat of the fire rather than use it with little effect, as is so often done. Such a method of operation would preclude the possibility of a single large connection menacing the entire community.

Some Notes on Street Cleaning

Among the many papers read last week before the annual convention of the American Society of Municipal Improvements, at Boston, Mass., there was one on street cleaning in general and a second, a very brief one on special practice with one kind of pavement at Providence. Those portions of the former having to do with machine cleaning are presented below in condensed form, and here is also given the substance of the notes from Providence.

Machine Street Cleaning

By Edward D. Very*

New York has 2173 miles of paved streets, Chicago 1899, Philadelphia 1371, and the other large cities have from 100 to 500 miles. Under average conditions a man can hand clean but about one-half mile per day. Even could he perform the work with thoroughness, the cost of such an army of labor for this work would be prohibitive. Machines at present used can do from two to four miles each and this work is done with a good degree of thoroughness, making daily work on the same area unnecessary. The patrol man then merely polices the streets as a litter gatherer and can cover a very considerably larger route.

The machine broom as today operated leaves much to be desired; purely, I think, because the action of the broom is dependent upon the speed of traction, whereas should a motor vehicle be substituted, power may be transmitted to the axis of the broom independent of the tractive power of the machine, a proper rate of revolution of the broom may be found and maintained irrespective of the speed of progress of the machine. At present if you find a particular filthy spot and endeavor to slow up to give it more careful attention, the speed of the broom lessens and the effect is loss, whereas were the power independent the result would be that required.

A mechanical device attached to the sweeping machine to pick up the stroke of sweepings from the broom and empty into a receptacle or receptacles would avoid a considerable amount of extra work now performed because the materials are spread by the wind or tracked by vehicle tires. Such machines are in evolution now but most of the devices are complicated or otherwise ill adapted to the performance of this work.

Flushing machines and squeegees are now constructed which do effective work but their adoption has been greatly retarded by the claims that the water has an adverse effect upon paving materials. My judgment is that the paving engineer should design his exposed surfaces to resist water action for the great benefit to be derived from water cleansing. In flushing machines there is a distinct advantage in having an attached pump to press the water so as to get a positive action continuously and on all the water which the storage tank may carry. A motor flushing machine has the advantage of carrying larger quantities of water with a consequent conservation of the time now lost in frequent filling of small tanks.

The machine squeegee or rubber scrubber is effective on smooth pavements where not too filthy, but the present method of delivery of water so close to the scrubber renders the work imperfect through the ineffective action of the water on the filth before the scrubbing is applied. This may be remedied by preceding the squeegee machines by a sprinkling cart sufficiently in advance to give the water action full effect. And here let me say that the sprinkling wagon should not be used, in my opinion, except as an adjunct to other cleansing devices, as by itself it is but a temporary makeshift and ineffective for good result. There is no reason for the filth removed by washing machines being deposited in sewer basins. The machine should be so operated as to leave the dirt near the curb, to be picked and carted away, as soon as it becomes sufficiently dry.

Dry-cleaning machines are greatly to be desired and good progress is being made in the development of such. I know of two types of vacuum cleaners which are doing good work and need but little to make them practical and effective. These, of course, should be designed to pick up and place in receptacles the filth removed. When dry cleaning machine work is adopted there must still be periodical wet cleaning and such a combination should insure the best results.

*Sanitary Engineer, 17 Battery Place, New York City; formerly of the New York Street Cleaning Department.

Clean Bituminous Pavements without Sprinkling

By Walter F. Slade*

In Providence we keep bituminous pavements clean by the patrol system, and reasonably free from dust. We believe that the use of water has an injurious effect upon the durability of the pavement. It emphasizes every slight depression and gathers and retains the fine particles that collect on a street, creating an unsightly appearance and a muddy, slippery condition. This results in the skidding of automobiles and a vast increase in the number of accidents.

By keeping the pavements free from water we escape another prolific source of complaint. Upon the best of pavements there will be depressions sufficient to retain a thin sheet of water. The rubber tires of the swiftly-moving auto, and especially the auto truck, will force the muddy water in a small stream upon the clothing of any one passing along the sidewalk, and even across the sidewalk, covering windows and buildings with muddy spots.

The Garbage of St. Louis is being dumped into the Mississippi River below the city. The Indiana Reduction Co. has a five-year contract for the treatment of the garbage but its plant was closed in July on account of objectionable odors. Another company has taken over the plant and will reopen it shortly, but the city is said to be planning to build its own garbage-reduction plant.

*Commissioner of Public Works, Providence, R. I.

Field and Office

Reinforcing the Winona Bridge, C. & N. W. Ry.

The Chicago & Northwestern Ry. crosses the Mississippi River at Winona, Minn., by a long bridge consisting of a series of through-truss spans. The bridge consists of 16 spans, with design and loading as follows:

(A) Twelve riveted lattice through-truss spans, built in 1886; five of 159 ft. 8 in., six of 142 ft. 2 in. and one (skew) of 144 ft. 6 in. Designed for a load of 3000 lb.

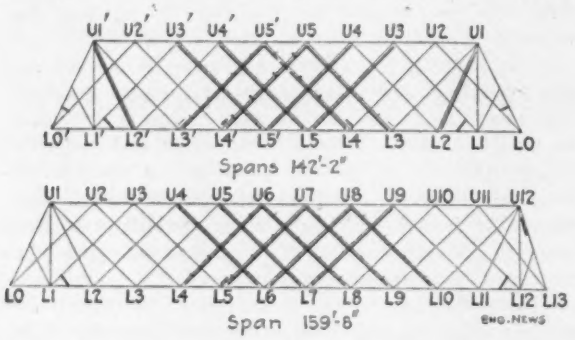

U1' U2' U3' U4' U5' U5 U4 U3 U2 U1
L0' L1' L2' L3' L4' L5' L5 L4 L3 L2 L1 L0
Spans 142'-2"
U1 U2 U3 U4 U5 U6 U7 U8 U9 U10 U11 U12
L0 L1 L2 L3 L4 L5 L6 L7 L8 L9 L10 L11 L12 L13
Span 159'-8"
ENG. NEWS

FIG. 1. RIVETED LATTICE TRUSSES OF THE WINONA BRIDGE; C. & N. W. RY.

(The heavy lines show the recent reinforcement.)

per lin.ft. with engine excess of 10,300 lb. Iron in tension, 8000 lb.; in compression, 8000 lb. for 159-ft. spans and 7000 lb. for shorter spans, reduced by Gordon's formula.

(B) Two pin-connected through-truss spans of 246 ft., built in 1893. Live-load, two 107-ton engines (2:8:0) followed by train load of 3600 lb. per lin.ft. Iron in tension, 10,000 lb. live load, 20,000 lb. dead-load; in compression, 7500 and 15,000 lb.

(C) One pin-connected draw span of 356 ft., built in 1898. Live-load, two 131½-ton engines (2:8:0) followed by 4000 lb. per lin.ft. Iron in tension, 9000 lb. for riveted members and 11,000 lb. for eye-bars; in compression, 8000 lb. reduced by Gordon's formula.

(D) One deck plate-girder span of 85 ft., built in 1898 for the same loading as the draw span.

With the increase in weight of locomotives and trains it became necessary to strengthen the 12 lattice spans to adapt them to the new conditions, and these spans have been reinforced to carry two 202-ton locomotives (2:8:0) followed by 4000 lb. per lin.ft. of train load. The other four spans, being designed for much heavier loading than the older lattice spans, and with low unit stresses, were found not stressed to excess by the modern engine loading.

A careful study was made of the composition and stresses in each span, and the reinforcement designed accordingly. This consisted mainly of additions to the web members of the trusses and the floor members of the riveted spans.

The trusses, Fig. 1, have some web members composed of only a pair of angles, and the principal reinforcement consisted in applying two additional angles to each overstressed member, attaching these to the original angles with tie plates. Additional connections to the chords were applied also, and in some cases struts were fitted to connect the end post with the first web member.

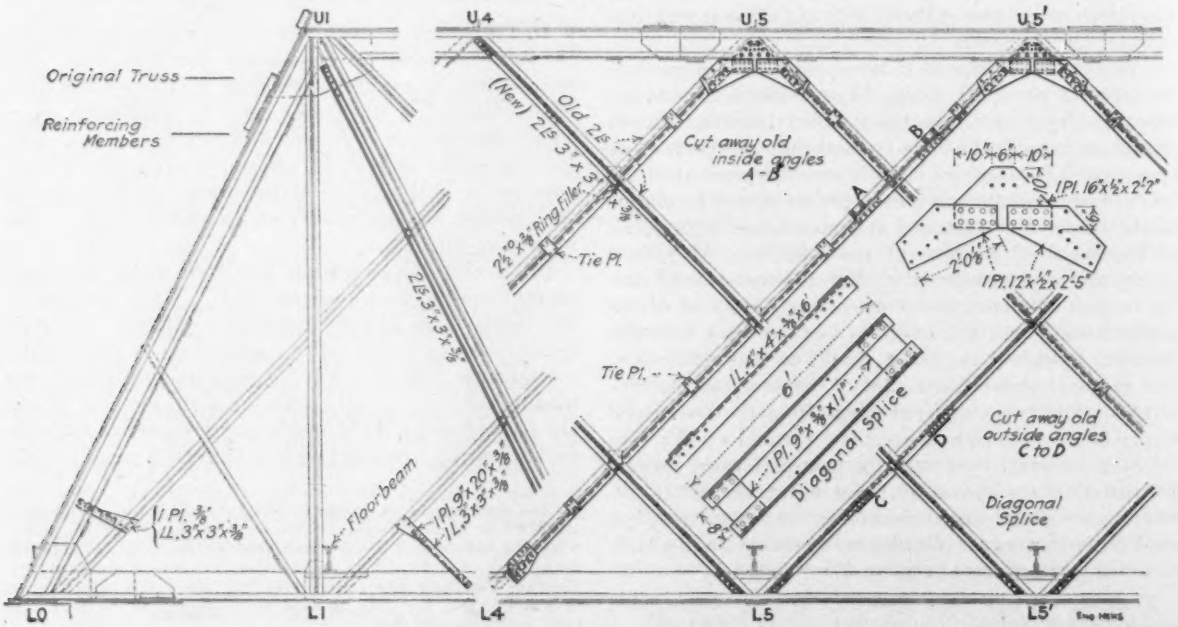


FIG. 2. REINFORCING OF RIVETED LATTICE TRUSSES

Fig. 1 shows the general design and the location of the reinforcement, while Fig. 2 shows further details of the work on the trusses.

In planning the reinforcement of the floor-beams, it was recognized that to strengthen them by adding cover-plates and other riveted work would be very expensive.

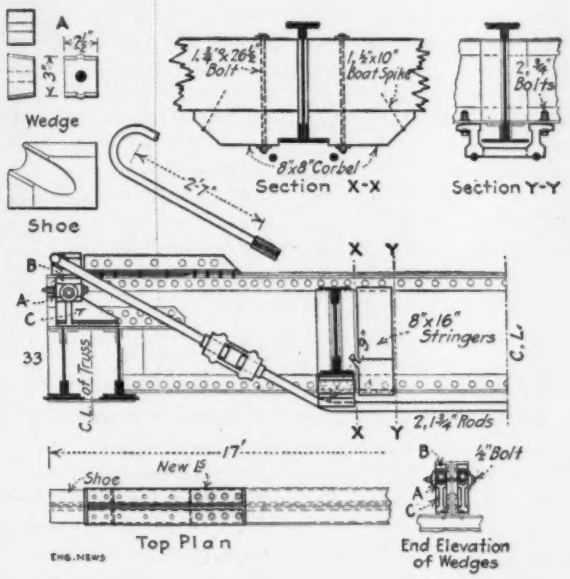


FIG. 3. REINFORCEMENT OF FLOOR-BEAMS BY TRUSS RODS; WINONA BRIDGE, C. & N. W. RY.

A method of securing the necessary additional strength by means of tie-rods was devised, therefore, and special aim was made to reduce to the minimum the amount of field drilling required for applying these rods. To reinforce the stringers and floor system a line of 8x16-in. fir stringers is placed inside of each of the original track stringers, their ends resting on corbels fitted to the bottom flange angles of the floor-beams.

The floor-beam reinforcement is shown in Fig. 3, and consists of a pair of 1¾-in. truss rods to each beam. Their ends are attached by turnbuckles to the legs of stirrups whose loop ends rest on shoes riveted to the top of the floor-beam at each end. The truss rods pass under cast-steel saddles located at the bottom flange of the floor-beam but bearing against the bottom flanges of the steel stringers. The bearings of the floor-beams on the trusses were strengthened by means of a set of cast wedges placed between the top and bottom angles of the floor-beams and driven to a bearing by means of a bolt. The device consists of the two opposite wedge pieces (A) between the cap (B) and pedestal (C). This arrangement was found to be much more economical and much more effective than riveting on short end stiffeners.

The method of carrying on the work under traffic was not difficult, as there was no occasion to obstruct the tracks, and the reinforcing of the trusses consisted only in light members which were easily hoisted into place. A gasoline air-compressor plant was installed at the east end of the bridge, and air pipes were carried on the floor to the various spans.

The work was done by the railway company's bridge department during 1913 and 1914. It was designed and executed under the direction of W. C. Armstrong, Engineer of Bridges, Chicago & Northwestern Ry., to whom we are indebted for our information.

Pile-Testing as Practiced with "Pedestal" Piles

Further discussion of the proper way to carry on a pile test or a soil bearing test* is contained in the following description of the MacArthur Concrete Pile & Foundation Co.'s practice in testing "Pedestal" piles (concrete piles with enlarged foot, formed in the ground). The so called pile formulas which are used for guessing at the capacity of a driven pile are altogether useless in the case of Pedestal piles, and therefore pile tests became almost indispensable. Hunley Abbott, chief engineer of the company, says that over 100 such pile tests have been made up to the present, and that, as these furnish experience and figures for almost every conceivable soil condition, tests are now not made usually except where the owner wishes them for his own satisfaction.

In preparing for a test the top of the pile is cleaned off and a concrete cap molded on it (see drawing, Fig. 1 herewith), the pile projecting a few inches into the cap. A steel bolt molded in the top of the cap furnishes a footing point for the leveling rod in measuring settlement.

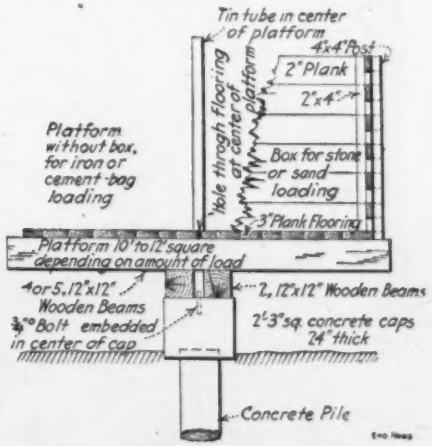

Tin tube in center of platform
4x4" Post
2" Plank
2"x4"
Hole through Flooring at center of platform
Platform without box, for iron or cement-bag loading
Box for stone or sand loading
3" Plank Flooring
Platform 10' to 12' square depending on amount of load
4 or 5, 12"x12" Wooden Beams
¾" Bolt embedded in center of cap
2, 12"x12" Wooden Beams
2'-3" sq. concrete caps 24" thick
Concrete Pile

FIG. 1. SKETCH OF PILE-TESTING RIG USED IN TESTS OF PEDESTAL PILES

On the cap is built up a platform composed of two tiers of 12x12 grillage covered with a 3-in. plank floor. If the loading material is sand or broken stone, a box must be built on the platform, but for pig iron or cement in sacks the simple platform will do. A tin tube passing through a hole in the middle of the floor enables the leveling rod to reach the bolt in the pile top.

Blocking near the ends of the 12x12 timbers keeps the platform from tipping while the load is being placed; but by the time the full load is on the load should have been brought to a balance, so that the blocking can be removed.

A level reading on the bolt should be taken before any weight is placed on the platform. Further readings

*For previous data on these two subjects see May 7, 1914, p. 1024 (Erie R.R.); July 23, 1914, p. 203; July 30, 1914, p. 250 (Panama-Pacific); Aug. 6, 1914, p. 310 (Pittsburgh); Sept. 3, 1914, p. 510; Sept. 24, 1914, p. 647; Oct. 1, 1914, p. 704 (Michigan Central).

FIG. 2. A 70-TON TEST LOAD ON A PEDESTAL PILE FOR A GAS-HOLDER FOUNDATION, NEW BEDFORD, MASS.

should be taken at various stages of the loading and a final reading when the load is all on. The total load should be allowed to remain on for several hours and readings taken to ascertain whether any further settlement occurs. Mr. Abbott's experience indicates that if the pile shows any settling tendency at all after receiving the final load, this tendency is relatively small, and the pile will come to a complete stop in a very few hours after ending the application of the load (24 hr. ample in all cases).

Molded-in-place piles should be allowed at least three weeks for setting of the concrete before load is placed on the pile, even under the most favorable conditions; better four weeks.

Fig. 2 shows a test made according to the procedure above described on a Pedestal pile installed for the foundations of a large gas-holder at New Bedford, Mass. The test pile was chosen at random by the engineers and all level readings were taken by them. No settlement was observed up to 40 tons loading; from 40 to 70 tons a small settlement developed, the total at 70 tons being less than $\frac{3}{8}$ in. after the load had been on for 24 hr.

Another interesting series of tests was made on Pedestal piles for the foundations of the Canadian Vickers plant at Montreal. Out of 3000 piles driven for this work, 19 were tested. Each was loaded to 42 tons standing for 24 hr. The total settlement ranged from $\frac{1}{32}$ in. to a scant $\frac{1}{4}$ in. In this case the loading was done with pig iron on a flat platform. In testing four piles in the foundations of the Subtreasury building, San Francisco, Calif., the maximum loading was 52 tons. No settlement was observed. These piles were tested with a cement-bag loading on a platform (without box).

Lining Large Coal Bunkers by the Cement-Gun Process

The new plant of the Minnesota Steel Co., at Duluth, Minn., has 33 coal bunkers for the gas-producers and a large circular storage bin for the coke ovens. It was decided to line these with cement mortar for protection against rusting. The lining was applied by the cement-gun process. It was intended originally to use a 6-in. lining of concrete placed by hand, but the cement-gun work was not only done at greatly reduced cost, but it also gave a lining of greater density and (owing to the impelling force) it insured closer contact and adhesion with the steel.

FIG. 1. LINING A 2000-TON COAL TANK WITH CONCRETE BY THE CEMENT-GUN PROCESS

(The men are applying the first coat, filling behind and over the steel reinforcement.)

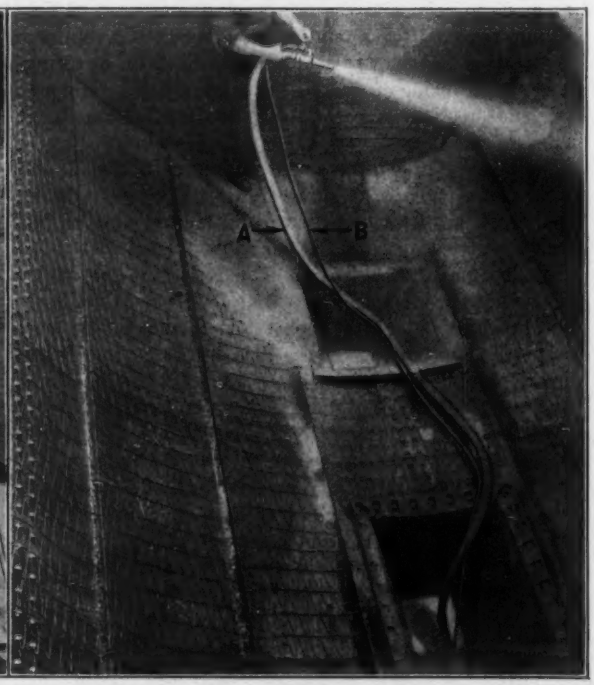


FIG. 2. LINING A COAL BUNKER BY THE CEMENT-GUN PROCESS

(A is the hose for dry concrete mix. B is the water hose, the water being added at the nozzle.)

The coal-storage bin which serves the coke ovens is 55 ft. diameter and 40 ft. high, with a capacity of 2100 tons, and is supported on a framing of steel girders and columns. It is fed at the top by a conveyor, while the coal is drawn off at the bottom by 20 hoppers 7 ft. square and 9 ft. deep, through which the coal is delivered to the charging cars. The capacity is equal to about one day's supply, so that the coal in the tank is practically in continual motion.

To the steel shell were riveted Z-bar lugs to which were fastened ½-in. vertical rods, and to these was wired a layer of triangular-mesh wire netting, about ¾ in. from the shell. The mortar is a 1:4 mixture of cement and sand; the dry mixture, forced through hose by the cement-gun, receives the mixing water at the nozzle from a separate pipe. In the impact upon the steel a portion of the sand is liberated, so that the actual mixture as placed is about 1:3½. The air-pressure in the cement-gun was about 50 lb. and the concrete was conveyed through a 1½-in. hose line to a maximum distance of about 200 ft., of which 100 ft. was vertical elevation. The concrete was applied in two coats, the first or stiffener coat filling behind the reinforcement and the second forming the finish. The total thickness is 5 in.

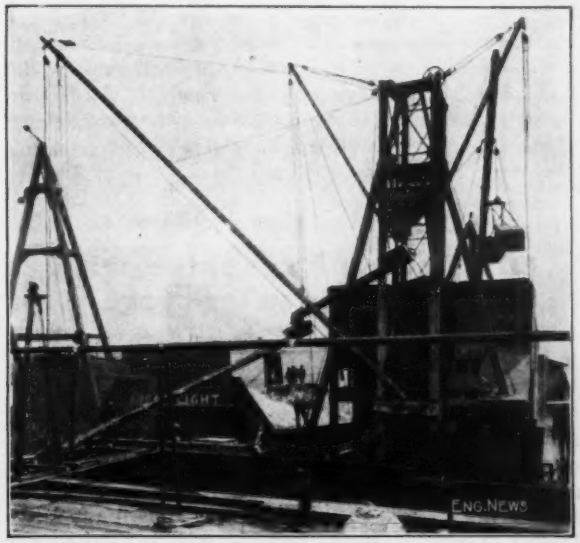


FIG. 1. THE FLOATING MIXER BUILDING A SEA WALL AT CLAREMONT, N. J.

(Showing chutes from tower to tremie; t and g creosoted permanent sheetpiling in lower foreground; bucket on floating plant feeds material hoppers; sand scow on left. Output of mixer 1 cu.yd. of concrete per min.)

The working crew consisted of a foreman and nine men, three of the men working on the scaffold in the tank; as shown in Fig. 1. The work occupied about six weeks working time for the application of the reinforcement and the lining.

The same lining, but with a thickness of only 2 in., was used for the 33 parabolic bins serving the gas producers. These bins are 19 ft. square and 12 ft. deep. Fig. 2 shows the work being done in one of these bins.

The work was executed under the direction of K. C. Hoxie, Chief Engineer of the Minnesota Steel Co. The Cement-Gun Construction Co., of Chicago, had the contract for the work of lining all the bins and bunkers.

Building a Concrete Sea Wall with a Special Floating Mixer

A rock-filled crib bulkhead at the Standard Oil Co.'s Eagle Works at Claremont, N. J., is being reinforced, practically replaced, by the construction of a heavy concrete wall in front. The crib is about 20 years old and has deteriorated to such an extent that the rocks were falling out at the face. The bulkhead is about 700 ft. long by 23 ft. high. The concrete wall, 10 ft. thick, was placed in water, by means of a tremie. A specially laid-out floating concrete-mixing plant was used.

FLOATING CONCRETE-MIXING PLANT

A concreting tower was erected on a scow, with mixer adjoining and arranged so that the concrete moved by gravity into the 1-cu.yd. hoisting bucket, as shown in Fig. 1. On one side of the floating structure was a sand scow, while a scow-load of gravel lay at the other. These material scows were unloaded into hoppers by the derrick shown forward on the mixer scow.

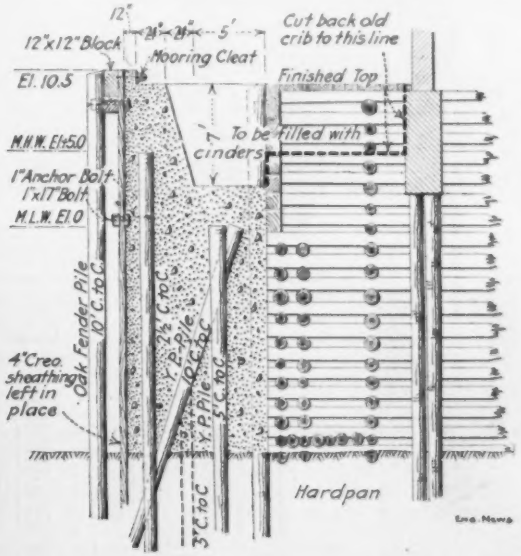


FIG. 2. CROSS-SECTION OF NEW-CONCRETE AND OLD-CRIB BULKHEAD

The rate of mixing was one batch (1 cu.yd.) per min. The materials were handled rapidly and easily, and the output of the plant limited only by the time required to mix. The first two lengths of chute were supported by booms extending from the tower. The third length was held by an A-frame placed on the new wall. The concrete passed from the chute into a hopper which fed the tremie. A pump supplied salt water for the mixer.

METHOD OF PROCEDURE

The site was first dredged to hard bottom, about 15 ft. below low water. Bearing and brace piles, 35 ft. long, were driven and cut off at low water. A line of 4-in. t & g creosoted sheetpiling to serve as face form was then put down, and the concrete deposited. The work was carried on in 100-ft. sections, one section being completed before another was commenced, so that the finished portion was available for commercial use.

The concrete was carried up full width to half-tide level (2 ft. 6 in. above low water) and above this the rear face battered as shown in Fig. 2. A 12x12-in. concrete backing log was placed along the top of the wall and provided with cleats for tying up boats. The timber crib was cut off 5 ft. from the top, and the space thus made as well as that back of the curb wall, filled with cinders. A plank floor was then laid over both the wall and the crib.

The creosoted sheetpiling will be left in place to protect the concrete against deterioration from tide action and temperature variation.

The work is being done by Geo. B. Spearin, 90 West St., New York City.

Calibrated Orifices at the Brooklyn Sewage-Experiment Station

By Geo. T. Hammond*

A considerable number of graduated adjustable orifices are being used to control the flow to the settling tanks, filters and other elements of the experimental sewage-treatment plant at the 26th Ward pumping station, Brooklyn, N. Y. Before installation, each orifice was calibrated in place by actual measurement of the discharge from one of the large tanks at the experiment plant.

Fig. 1. View of Orifice

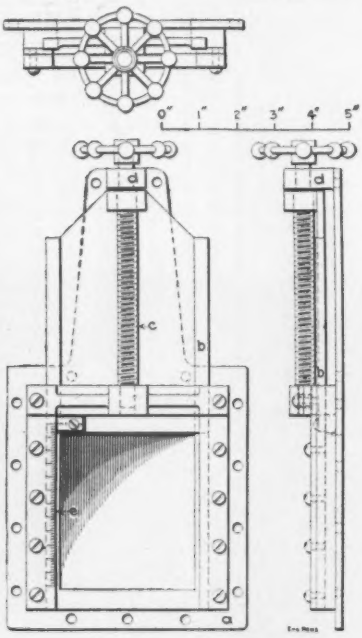


Fig. 2. Detail of Orifice

Calibrated Orifice, Brooklyn Sewage Experimental Plant

The sewage at the experiment station is first pumped to a quieting tank and is then passed through one or the other of six orifices to a flume leading to a unit of the plant. When the effluent from one of these units receives further treatment it is again passed through an orifice, placed in an orifice box, to control the rate of flow.

The orifices were supplied, installed and calibrated by the Wallace & Tiernan Co., Inc., of 136 Liberty St., New York City. The smallest of the orifices called for by the specifications was required to measure flows ranging up to 150,000 gal. per 24 hr. and the largest up to 600,000 gal.

Fig. 1 shows one of the orifices in use and Fig. 2 shows one in detail. In the drawing *a* is a back plate, screwed to the outside of the constant-level box; *b* is a sliding gate moved by screw *c*, which turns in the threaded part *d*. The scale *e* is made by obtaining points by actual measurements of the sewage discharged through the orifice at different openings into the tanks of the plant; several points on the scale were thus obtained and the others interpolated.

The orifices are of bronze. All sliding parts are machined so as to work easily and yet fit tightly. They are made according to the principle of hydraulic discharge; but the discharge as actually measured varied somewhat, owing probably to the velocity of approach through the 2-in. planking of the wall of the constant-level boxes on the outside of which they are placed.

E. J. Fort is Chief Engineer of the Brooklyn Bureau of Sewers.

*Engineer of Design, Bureau of Sewers, 215 Montague St., Brooklyn, N. Y.

Oil-Engine Excavator with Detachable Tractor

The Turner Excavator, used in the emergency ditching job described on p. 741 of the Oct. 8 issue of Engineering News, is a strikingly new type of machine. The excavator and tractor parts are made separate, each with two wheels, their frames being coupled together somewhat like the coupling of motor-truck tractors and trailers. Moreover, the forward part of the machine, i.e., the excavator portion, can be coupled up with any other tractor or traction engine. The machine is built by the United Iron Works Co., Springfield, Mo.

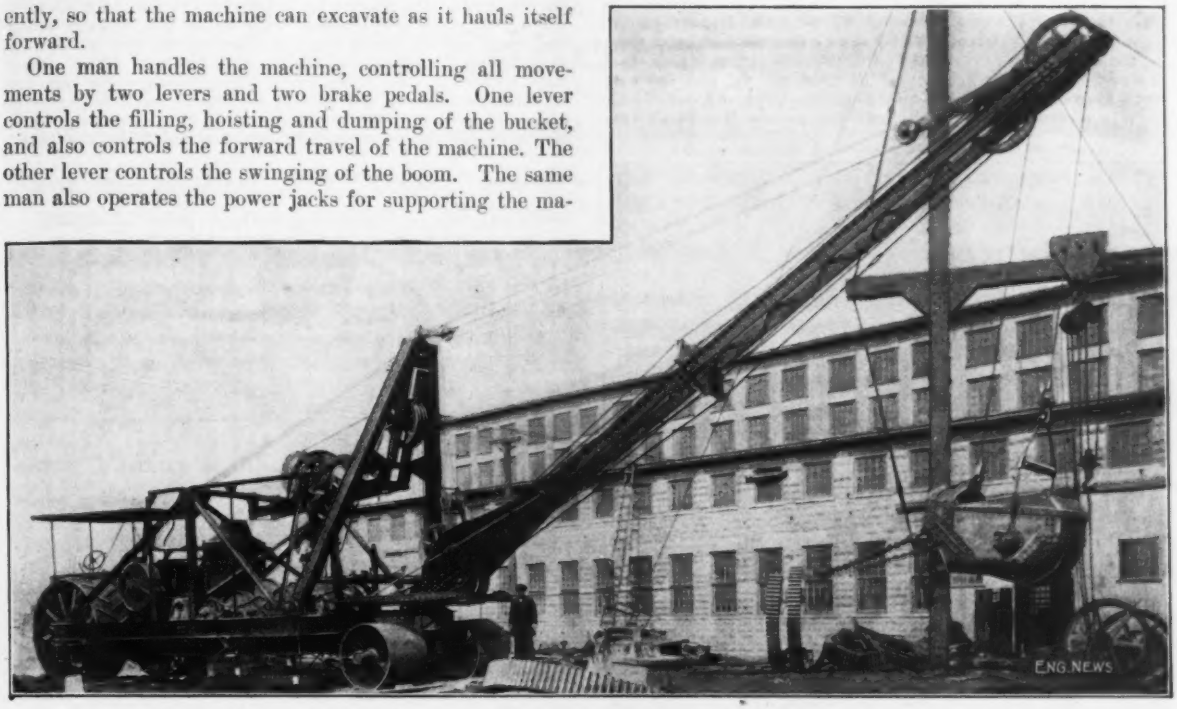
The machine is primarily a drag-line machine, but it can be converted into a steam shovel by some changes: replacing the long boom (see view) by a shorter plate-girder boom with dipper handle and bucket.

The excavator machinery is mounted on a trussed-steel frame, with a pair of wide-tired wheels under the front end. The rear end of the frame is forked, and embraces the forward part of the tractor, the latter resting upon a crossbrace of the frame. The sills extend back far enough to form outboard bearings for extension hubs which are bolted to the centers of the rear (driving) wheels. In adapting the excavator to an existing tractor, the front axle and wheels of the latter are removed, and its front end is supported on and attached to the crossbrace, as noted above. Any kind of tractor may be used (steam, oil or gasoline) and any kind of grab or scraper bucket can be handled.

The machine shown in the cut is 28 ft. long, with 40-ft. boom, 1½-yd. scraper bucket, an A-frame 15 ft. 3 in. high, and front wheels 42 in. diameter. It is driven by a 60-hp. oil tractor and weighs complete about 40 tons. In addition to its propelling gear it has a cable-towing drum which enables the machine to be operated readily over wet ground. The towing rig is driven independ-

ently, so that the machine can excavate as it hauls itself forward.

One man handles the machine, controlling all movements by two levers and two brake pedals. One lever controls the filling, hoisting and dumping of the bucket, and also controls the forward travel of the machine. The other lever controls the swinging of the boom. The same man also operates the power jacks for supporting the machine on either side, and shifts the platform tracks used on soft ground. These platforms are 18 ft. long, and much wider than the wheels.



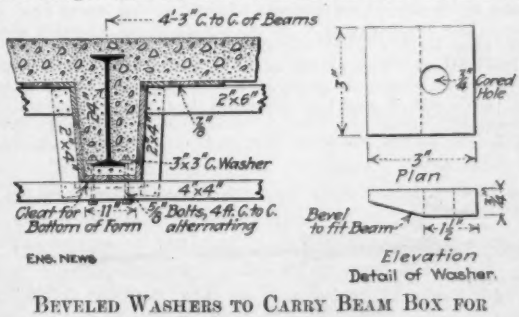
DRAG-LINE EXCAVATOR OPERATED BY AN OIL TRACTOR OF 60 HP.
(Union Iron Works Co., Springfield, Mo., builders.)

NOTES

A Beveled Washer as the main detail of a form support for forms for placing concrete incasement around steel beams was used by Peters & Kiehm, the contractors for the Pleasant St. viaduct of the Delaware, Lackawanna & Western R.R. at Utica, N. Y. The detail of the construction is given in the accompanying cut. This part of the bridge consisted of 24-in. longitudinal I-beams spaced 4 ft. 3 in. center to center, completely incased with 2 in. of concrete and carrying the roadway on a reinforced-concrete superposed slab.

The form consisted of 4x4-in. stringers hung 4 ft. on center from the lower flange of the main I-beam, carrying the concrete form as shown in the drawing.

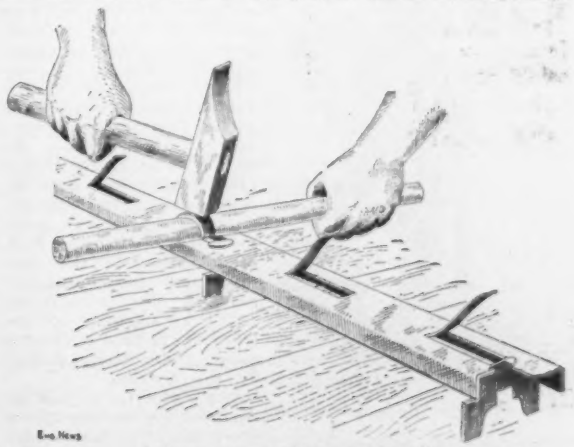
The hanger bolts had their head at the lower end and were hung from the lower flanges of the I-beam by specially made washers beveled as shown in the detail on the drawing to fit the lower flange of the beam. In erection, the 4x4-in. sticks were hung on one set of bolts, from one flange of the beam, and the bottom of the beam box placed on the opposite side, the bolts being put through holes bored for that purpose. Then the first placed bolts were removed, allowing the beam box to be hung by the second set of bolts, while the first line of hanger bolts was replaced and the beam box sides and floor forms were set.



BEVELED WASHERS TO CARRY BEAM BOX FOR CONCRETE INCASEMENT

The bolts and nuts were well greased and after the concrete was placed the forms were taken down by unscrewing the heads of the bolts, leaving the washers and the nuts in the concrete; the holes in the concrete were plugged up with cement mortar.

A Patented Spacing Bar for holding and spacing reinforcing rods in concrete slabs has just been put on the market by the Electric Welding Co., of Pittsburgh, Penn. The accompanying sketch gives a view of the bar and the method of its use. It is known as the "Efficiency Spacing Bar." The bar consists of a ¾-in. channel pierced at proper intervals for floor rod spacing (say 3 or 4 in.) with openings the metal strip from which is bent up at an angle sufficient to admit the floor rod. At every other one of these openings a chair is welded so as to carry the spacing bar (and the reinforcing rods) the proper distance above the bottom of the slab. The floor rods are clamped solidly into place by a hammer blow, which easily bends the projecting metal strip tight over the rod.



"EFFICIENCY SPACING BAR"

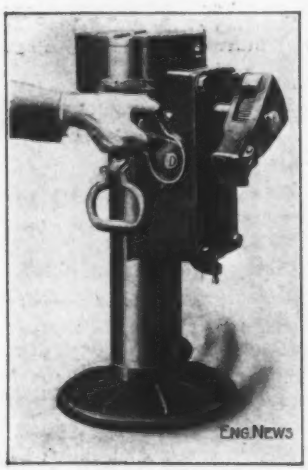
The section of the bar is about ⅝ sq.in. so that it serves also as a transverse stiffening or reinforcing rod.

Dumping a Wagonload of Old Brick—The accompanying view taken in the new Western Yard of the Department of Public Works of Detroit, Mich., shows the method used there in dumping discarded brick onto the pile from which they are passed into the crusher to be used for fill and for concrete aggregate. The wagons, holding about one cubic yard each, are driven into the yard, the horses unhitched, the

LIFTING A TWO-HORSE WAGON OVER A BRICK DUMP

wagon picked up by the special chain sling shown and lifted up above the pile; then the driver throws the bottom dump and the wagon is returned to the waiting horses in something over a minute. The method localizes the dump pile because it permits much higher piling than would be possible by the ordinary dumping methods.

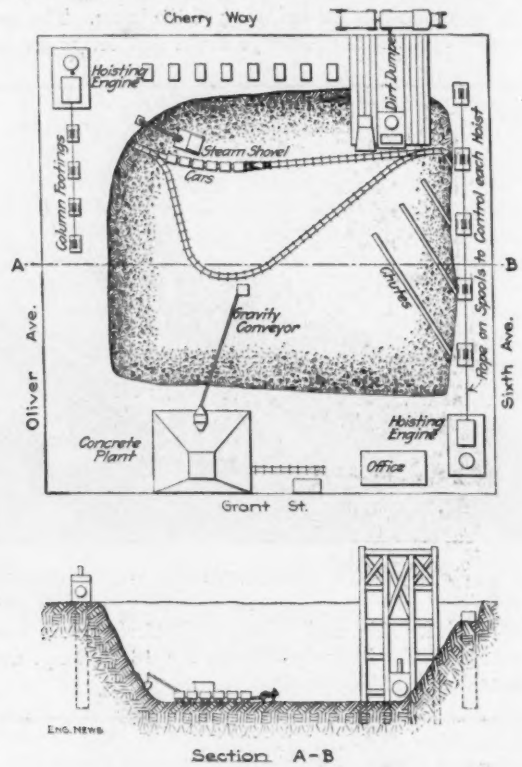
Screw Jacks With a Friction-Brake Lowering Device are now being built by A. O. Norton, Inc., of Boston, Mass. The friction-brake device makes it safe to lower the load under all conditions. It consists of a high-multiplication gear train (driven by the stem gear only when lowering), a friction disk at the outer end of this train, and a thumb-lever to release the spring friction which bears on this disk. When the jack is to lower, the lifting ratchet is thrown out; the main screw of the jack, being of steep enough pitch to be rotated by the load, is now held by the gear train and the friction; by pressing the thumb-lever and thus slacking off the friction, the load can lower away, under ready control by the friction.



BALL-BEARING SCREW JACK EQUIPPED WITH FRICTION-BRAKE LOWERING DEVICE

The claim for a jack so equipped is its safety in operation. The friction device has been applied to 25-, 35- and 50-ton jacks, in both the head-lift and the foot-lift patterns.

The Excavation Plan of the William Penn Hotel, at Pittsburgh, Penn., was described with some views in "Engineering News," Oct. 8, 1914, p. 745. The accompanying cut, reproduced from an unscaled sketch, made at the site, shows the general layout of the work. A steamshovel dumps into narrow-gage cars drawn by horses to the dumping elevator where the cars are emptied into skips which drop below the track level. These skips are pulled up the incline and dumped into automobile trucks and trailers which empty onto barges at the river front. The footings for the line of columns next to the streets are being sunk in timbered open pits from the high level, as shown in the section. These pits are controlled by buckets hung to spools over each pit



LAYOUT OF PLANT FOR EXCAVATING; WM. PENN HOTEL

and on each street a continuously running line passes from a hoisting engine over each pit. The pit spools are then started, when required to lower or elevate buckets, by throwing in a clutch. The footings are concreted from a gravity tower, fed from the mixing plant on the Grant St. side of the lot. The contractor is James L. Stuart.

A Guy Anchor Without Loose Parts—Successful trials were made of a new guy anchor having no loose members, and the guy anchor is now being manufactured for the market by the Barbed Anchor Mfg. Co., Paul Jones Bldg., Louisville, Ky. As the adjoined sketch shows, the anchor consists of a straight bar forming the body of the anchor or the drive point, and two sets of bent spring barbs near the point. The main rod is a ⅞-in. square piece of steel about 5 ft. long with a 1-in. eye forged at the top and a point at the bottom. The barbs, each of which is backed with a second strip to give it more stiffness, are riveted to the rod. The rod itself is painted, whereas the bars and rivets are galvanized; since the rod is proportioned for driving it has ample section to stand the pull of the guy even after

BARBED DRIVE-ANCHOR

losing a large amount of metal by rusting. The anchor complete weighs about 20 lb. and costs something over $1.

In one test of the anchor, where it was used to guy a pole, block and tackle were used to pull the anchor up to bearing after it was driven. The upward motion was about 6 in. Then the strain line (a guy to a pole-line deadend) was attached; in the first week the anchor gave ⅜ in., but after that showed no give.

Editorials

Status of the Proposed International Congresses at San Francisco in 1915

The first positive action regarding the International Congresses, scheduled to be held in San Francisco next year during the Panama-Pacific Exposition, is the announcement that the Electrical Congress will be indefinitely postponed. The Executive Committee has been continued, its records preserved, and the bills will be settled by the American Institute of Electrical Engineers. It is intended to hold the congress eventually, and in America.

In view of the absolute impossibility of holding a truly international meeting, on account of the European War, this action is entirely wise. Those organizations which carry through their previous plans for holding congresses may possibly secure a fair American attendance, although even that is problematical in view of the decreased interest which will result from the scanty foreign representation, and because many American engineers will be deterred by business conditions from taking the time and money necessary to attend a congress at San Francisco next year.

Ethics and Compensation of Engineers Again

In our issue of May 7 last, we published a letter entitled, "Ethics and Compensation of Engineers from the Standpoint of a Lawyer." The second paragraph of this letter read as follows:

> **An active writer on engineering ethics, prominent in the American Society of Civil Engineers, who would unquestionably consider it unethical for an engineer to guarantee or assume the financial responsibility for the commercial success of a new design in practice, had no scruples about reporting under his signature, on the same day, that the same railroad project would cost $17,000 per mile in his report to the promoting company, and $25,000 per mile in his report to the Provincial Ministry, for the purpose of securing a subsidy for the promoting company of $20,000 per mile.**

The fact that no names were mentioned in the above quotation or in the letter of which it formed a part is probably the reason why it passed editorial scrutiny, although our scrutiny ought to have been particularly strict since the letter purported to be written by a lawyer! It is part of a lawyer's business to state facts in such a manner that they will produce a desired impression. Since the letter was published, statements have been laid before us which show that the statement above quoted was entirely unwarranted.

The papers in the case show that the engineer in question did present two different estimates of cost for a projected railroad, and in fact presented more than two different estimates; *but the different estimates were based on widely different specifications.* One estimate, for example, was based on entirely different requirements as to maximum grade and curvature, weight of rails, etc., from the other. It should be further stated that both reports were made to the promoting company, the engineer's client, and that no report was made to the Provincial Ministry, save that the engineer in question represented his client in negotiations over the railway project, which later became the subject of a bitter political controversy in the Provincial Parliament; and as those familiar with Canadian politics are aware, professional and personal and business reputations are often sacrificed to the necessities of partisan political squabbles.

In the present case, however, the papers submitted to us indicate that even the attorneys of the opposition conceded at the end that the action of the engineer in question was that of a high-minded professional man discharging his duty toward his client. It was very likely some distorted newspaper report of the squabbles in the Provincial Parliament which led to the unfounded accusation in our issue of May 7, referred to above.

We deem it worth while, even at this late date, to set forth the facts in this case in order that at least one unjust and unwarranted attack upon the honor of the engineering profession, as represented by one of its prominent members, may be rendered of no effect.

Military Engineering Training for the Engineering Student

We believe that engineers who were educated at a "land-grant" college generally look back upon their compulsory military training there (which is the federal government's compensation for financial assistance) as a period of drudgery. While the young male is essentially a military creature and the tin soldier a plaything *par excellence* to the boy, the soldier's life, or at least that part of a soldier's life which involves two hours' drill inside of a dusty and ill-ventilated armory, does not as a rule appeal to the more mature boy at college. However, an all-wise government considers that even this limited military experience is a desirable part of the education of young men; and furthermore it is constantly trying to make this enforced military drill somewhat more interesting to the students It is worthy of note in this connection that special instruction is now being given in military engineering in some of the land-grant colleges.

Under the American system of army development, the United States must depend for its armed forces in time of war upon a small nucleus formed of the regular army, distributed as a leaven through the great mass of volunteer troops. In order that those who may have to make up this volunteer army should not be entirely ignorant of the work which they may be called upon to perform, certain of the youth of the country are trained in the elements of military science either in school, in college or in the National Guard. In the National Guard, there are sections devoted to the practice of military engineering, which perform the functions of the engineer-companies of the regular army. These sections, however, are much too small and the time given to their work much

too limited to take up all of the duties which naturally devolve upon an engineer corps in time of war. In fact, in the Spanish-American War there were formed some volunteer engineer companies.

In the event of any extensive war, such companies must be created and their efficiency will naturally be greater if they are made up of men whose ordinary occupation has to do with engineering. Yet while such civilian engineers are eminently more fitted for the practice of military engineering than would be a civilian who knew nothing of engineering, the entirely novel features of military engineering are so many as to require considerable time and labor on the part of a civilan engineer if he is to be capable of performing the work of his military brother.

It seems eminently fitting, therefore, that boys engaged in the study of civil engineering, who are required by the government to devote some time to military studies, should devote that time to the study of military engineering. Thus in the event of a war where their services would be required, they would be able to bring to the very important engineering corps of the army not only a training in civil engineering but some elementary knowledge, at least, of the military engineer's work. At the same time it adds to the student engineer's curriculum a study which should be much more interesting than the routine drilling of the infantry branch.

At Cornell University during the coming year an engineer corps is to be added to the student "army," and is to be not only under the direction of the U. S. Army officer who is in charge of military instruction there, but is also to have instruction by a retired sergeant of engineers of the U. S. Army, a man who can undoubtedly bring to the instruction of the student more practical experience than the line officer who is commandant.

The Design of the New York Barge Canal

We quoted in our issue of Aug. 27, 1914, a statement by John A. Bensel, State Engineer of New York, to the effect that a Western manufacturing concern which desired to use the New York Barge Canal when completed and investigated the relative cost of rail and water transportation found that for economical transportation freight should be shipped through without transfer. The ship designers, however, Mr. Bensel stated, have not yet been able to design a vessel which could navigate both the Lakes and the barge canal with the limited depth and overhead clearance prevailing on the canal.

This statement of Mr. Bensel's was discussed in a later issue (Sept. 10) by John Reid, the well known marine architect, and he raised the very natural question why the barge canal was not designed in the first place so that it would accommodate vessels capable of Lake navigation. Mr. Reid said:

> It has always seemed extraordinary to naval architects and shipbuilders that not the slightest attempt seems to have been made in designing the New York Barge Canal to decide beforehand what types of boats would be the most suitable and the most likely to be built for use thereon. Mr. Bensel, in his statement, declares that shipbuilders have found it impracticable to design a steamer suitable for navigating the Great Lakes and also for the navigation of the Barge Canal. The question naturally arises why was not the Barge Canal originally designed so as to be adapted for the passage of vessels which could navigate the Great Lakes?

It is proper that this question should be brought up and answered in these columns since it is a question which will doubtless receive wide public attention as soon as the canal is ready for traffic. The commercial interests of the State of New York when they inquire what benefit they are to receive from the enormous expenditure which the State of New York has made in the construction of this waterway will be very likely to jump at the conclusion that a stupid blunder was made by the engineers who planned the canal in not adapting it to vessels which could navigate Lake Erie at least, so that cargoes could go through without breaking bulk from any of the Lake Erie ports to tidewater.

It was not, however, by a blunder that the canal was built in this manner. On the contrary, strange though it may seem, the New York Barge Canal was actually designed with a distinct purpose to prevent its navigation by vessels capable of traversing the Lakes and so that all cargoes would have to be transferred at Buffalo.

To explain how this came about it will be necessary to review briefly the history of the Barge Canal project.

Twenty-five years ago, in the late '80's and early '90's, great interest was aroused in the States bordering on the Great Lakes over the project to extend Lake navigation to the Atlantic seaboard. At that time the grain and flour trade on the Lakes was the largest item of traffic. The iron ore and coal business was of small proportions compared with what it has since become. Railway rates from the Central States to tidewater, on the huge volume of export grain which was then moved to the seaboard were comparatively high. The Erie Canal had lost its former commercial importance and it was clearly recognized would soon become obsolete as a traffic route. Commercial interests were agreed that a water route to take the place of the Erie Canal and designed in accordance with modern traffic requirements was a necessity of the future.

In 1884, the late Elnathan Sweet, a former State Engineer of New York, read a paper before the American Society of Civil Engineers outlining a project for the rebuilding of the Erie Canal as a ship canal capable of carrying vessels of 18-ft. draught. Interest in the Lake cities became more and more active and various conventions were held to further the survey and construction by the United States of a deep waterway from the Lakes to the Atlantic.

In 1895, in response to this agitation in the Lake states, an International Commission was created by joint action of the United States and Canada to study the best route for a deep waterway between the Lakes and the Atlantic. The American members of the Commission were James B. Angell, then President of the University of Michigan; John E. Russell, of Massachusetts, and Lyman E. Cooley, the well known engineer of Chicago.

This Commission was given only a small appropriation and was able to make only a preliminary investigation, but it reported that such a waterway was feasible and recommended a large appropriation sufficient to make complete surveys and estimates. Congress therefore made appropriations for such surveys aggregating nearly half a million dollars and President McKinley appointed as a Commission to conduct these surveys, Major Chas. W. Raymond of the Corps of Engineers, Alfred Noble and George Y. Wisner.

As most of our readers well know these three engineers, all of whom have now passed away, were among the most eminent men in the profession; and their conclusions and recommendations were entitled to great weight. The

Commission spent three years in its work and made complete surveys and estimates for both a 21-ft. deep waterway and a 30-ft. deep waterway from Duluth and from Chicago to tidewater by various routes. It recommended the 21-ft. channel as the most advantageous commercially. The estimated cost of its construction was in round numbers $200,000,000, of which $155,000,000 was the cost of the section from Lake Ontario to tidewater, across the State of New York.

But the commercial interests in the Lake cities which six or seven years earlier had agitated so vigorously for government construction of a deep waterway between the Lakes and the Atlantic paid no attention whatever to this report, when it was presented in 1900. A partial explanation is that the Lake carriers who had a few years earlier been anxious to have an outlet to the Atlantic for their fleet now found themselves taxed to their capacity in caring for the ore and coal traffic to the head of the Lakes. But the more important reason, doubtless, for the shelving of the report and abandonment of the project was that the State of New York had by this time determined to provide its own waterway to the Great Lakes by enlarging the Erie Canal.

While the Board of Engineers for Deep Waterways was at work on its surveys, Theodore Roosevelt, who was then Governor of New York, appointed a commission in 1898 to advise what New York should do with its canal system. The Erie Canal, as has been stated above, was rapidly becoming obsolete and was no longer an important commercial factor. In 1895, the State had made an absurd attempt to do something for the canal by borrowing $9,000,000 to enlarge some of the locks and dredge to greater depth some of the canal levels. The expenditure of this $9,000,000 appropriation was accompanied by scandals which nearly wrecked the administration. It was evident, however, that even if the money had been honestly and efficiently expended, the outlay of any such trifling amount on this shallow old canal was practically wasted.

The Commission which Governor Roosevelt appointed was headed by General Francis V. Greene, a well known engineer. The report of this Commission, presented in 1899, practically ignored the Federal project for a deep waterway from the Lakes to the Atlantic, then under survey and recommended that New York should with its own funds undertake the construction of a waterway to supersede the Erie Canal. The Commission advised that such a waterway should be made large enough to accommodate barges of 1000 tons capacity; and the canal which was afterward undertaken has been generally known as the 1000-ton barge canal. The work has been carried out to a large extent on the lines laid down by that Commission's report, save that the dimensions have been increased so that, in theory at least, barges of a capacity of as much as 3000 tons, will be able to pass through the canal.

The interesting point, however, is the consideration which caused the Commission and the legislature which acted upon its recommendations to fix upon a canal of this type. It should be understood that the only two sections of the State which took any great interest in the canal project were New York City and Buffalo. Other sections of the State, even along the canal line, were either lukewarm or strongly opposed to the assumption of a huge debt by the State for this purpose. New York City, clinging to the old tradition that her greatness was due to her location at the mouth of a waterway from the West, would have been satisfied with any waterway of large capacity, that projected by the Federal Government commission, for example. Buffalo, however, which had really fathered the project for enlarging the Erie Canal, was insistent upon one thing: that the waterway built should not be of such capacity that the lake traffic could pass her docks without stopping. What Buffalo wanted was a waterway across the State which would move traffic at the lowest possible cost and yet would not be large enough to admit vessels capable of navigating the lake!

It seems strange indeed, as we look back upon it now, that this huge engineering work—the greatest piece of waterway construction ever undertaken in the United States and probably the largest piece of public work ever undertaken as a State enterprise—should have been deliberately planned, not with a view to give the greatest public benefit, or even the greatest benefit to the State itself which paid for the work, but with the distinct object of protecting the commercial interests of a single city of that State.

Such, however, is the absolute fact. Buffalo's interest in the Barge Canal was as a means of building up her commerce and increasing the value of her real estate. Any proposal for permitting the Lake commerce to pass her doors without paying tribute, she would not hear of for an instant.

The mistake New York was making in adopting plans for a canal which would accommodate nothing but barges, instead of a channel which could be used by lake and coastwise vessels also without transferring cargoes at the terminals, was pointed out again and again in the columns of ENGINEERING NEWS. Attention was called to the fact that in view of the interest which had been taken by the Lake States in the project for a deep waterway between the Lakes and tidewater, New York could easily secure their coöperation and probably have the entire work undertaken by the Federal government or that it could at least do so if New York would offer to bear half of the expense, which would have made a much smaller burden for her than that which she shouldered in constructing the barge canal. At one time, as our older readers may recall, we published a symposium presenting the opinions of a dozen engineers standing at the head of the profession, all agreeing as to the unwisdom of the project which New York was entering upon.

We deem it proper to review this past history here, because it will undoubtedly be said that the engineers were to blame in not designing the New York Barge Canal for economic transportation. It is well to have on record the fact that it was not lack of engineering foresight or knowledge but the desire to further special interests which determined the type of New York's $100,000,000 waterway.

It is fair to add, however, that some economic facts which bear upon the problem were less widely known fifteen years ago than they are today. If those who represented Buffalo, for example, in furthering the canal enterprise, had understood as they would today the enormous expense involved in transferring freight compared with the cost of hauling it, they might have been less insistent that the canal must be so built as to compel transfers at Buffalo, since they would have realized the danger that a traffic route thus handicapped would very

likely prove unable to attract any large volume of traffic in competition with other routes.

Further than this, as was shown in these columns some weeks ago (ENG. NEWS, July 9, p. 90), the passage of commerce through a city, even where a transfer is involved, is nowadays a very small factor in the city's growth and prosperity. Galveston has after New York the largest foreign trade of any port in the United States; yet Galveston has only 37,000 population, while Buffalo is a city of over 400,000. Buffalo's commerce would benefit far more from a canal able to pass the Lake freighters than from the present canal which requires a transfer of cargo.

Subject to the limitation above noted it was endeavored to design the barge canal as an economic waterway. The canal locks were made phenomenally large for a barge canal because at that time towing of a fleet of barges by a single power boat was in great favor as an economical system of transport, especially on the Lakes. The evolution of the past fifteen years has been away from that system and toward the practice of giving each vessel its own motive power.

Probably the most serious handicap of all to the use of the new canal by vessels which could also navigate the lakes, is the numerous fixed bridges across the canal with a clear headway of only 15½ ft. This is an even more serious restriction to the boat designer than the limiting depth of 12 ft. If movable bridges instead of fixed bridges had been built over the canal, it would be possible to design a roomy freight carrier with shallow draft and very broad beam and to obtain a fair degree of longitudinal strength in its hull notwithstanding its shallow draft, by giving it a very high freeboard. Such a boat would be only fitted for slow speed, it is true; but it would be seaworthy enough to operate with reasonable safety on the lower Lakes during most of the season. The low headway under the fixed bridges, however, seems to shut off any possibility of the use of such vessels and restricts the use of the canal to low barges and tugs.

Of course, it will be freely admitted that to interpose possible interruptions to railway and highway traffic by a line of drawbridges over the canal extending from east to west clear across the State of New York would be a very serious matter. If, however, New York desired a great commercial waterway to extend across its territory, that is part of the price it would have had to pay.

As an alternative, a deep waterway could have been built at no greater cost by the St. Lawrence River and Lake Champlain route; and now that Canada with commendable enterprise is enlarging the Welland Canal to accommodate the largest lake steamers and is likely to enlarge the St. Lawrence canals eventually, either that or the route wholly through Canadian territory to the Gulf of St. Lawrence, is the probable future outlet for the Lake shipping to the Atlantic.

That the New York Barge Canal with its compulsory transfers at each end could not compete as a through traffic route with such a waterway is obvious.

Letters to the Editor

Trade Catalogs Wanted by South American Engineers

Sir—I should be glad if you would insert a paragraph in your widely read journal to the effect that The Institution of Engineers of the River Plate, of which I have the honor to be President, would be glad to receive and file for reference of members catalogs of engineering materials which manufacturers may desire to introduce to this market. The address of the Institution is 25 de Mayo 195, Buenos Aires, S. A.

BERTRAM C. HUME.

Field Test Values for Concrete

Sir—The article entitled "Field Tests of Concrete for Municipal Work at Kansas City, Mo.," in ENGINEERING NEWS, Sept. 10, 1914, p. 526, has just come to my attention.

I regret that these otherwise interesting tests are practically valueless because rubber pads were used for cushioning the ends of specimens in the testing machine. When the rubber comes under pressure it spreads laterally and introduces tensile stresses which split the end of the test specimen before the true compressive strength of the concrete is attained. The illustration accompanying the article in question shows that this happened to the specimens there reported. So far as can be determined from the illustration, all the cylinders are fractured through one of the ends. In properly manipulated tests the fracture either occurs in the middle of the cylinder, leaving the ends intact, or it originates in the middle and spreads to the ends, the fracture of the ends being a secondary result.

Your editorial on the subject is excellent, but in view of what has just been stated you have shown no satisfactory reason for reducing to a lower value Mr. Aiken's figure of 1500 lb. per sq.in. for 1:2:4 concrete at 30 days. The experience of the writer with a large number of field tests in New York City agrees very well with this figure.

RALPH E. GOODWIN.

Public Service Commission Testing Laboratory,
38 Park Place, New York City.

On Living Conditions and Commercial Opportunities in the Argentine

Sir—Replying to M. J. Lorente's letter in your issue of Oct. 15, 1914, I will say that while in Buenos Aires I bought several pairs of shoes for less than 14 or 16 paper dollars a pair, and being intimately acquainted with the representative of the United Shoe Machinery Co. who was installing many shoe-making machines in the

Argentine Republic, I can safely assert that at that time first-class native-made shoes could be bought for less than 7 or 8 U. S. dollars. Also I bought suits made to order for 85 and 90 paper dollars, which is equivalent to 36 and 38 U. S. dollars, respectively, and these suits were equal to those costing the same price in New York City.

I do not wish it to be understood to mean that French influence predominates in every enterprise in the country; and if Mr. Lorente will read again my letter of Sept. 5, 1914, he will find the statement that, "South America has in the past secured its capital in Europe . . . and for this reason the English have large interests there."

It is true that in business circles the "British influence has been infinitely more powerful than that of the French," but not so in the matters of tastes, culture, methods of living and the like.

British enterprises are quite naturally managed in accordance with British customs and their structures built according to British standards. This is also true of other nationalities having holdings in the republic, and I do not dispute the fact that British holdings in the country are very large and numerous. Neither do I wish to belittle the vast interests held by the Germans, for the writer has referred to the German "aggressiveness" and its result.

It must be borne in mind that Buenos Aires is a cosmopolitan city and that practically one-half the population is of foreign birth, and about one-fifth are Italians. It is therefore to be expected that to a certain degree the Italian influence is felt. Engineers of various nationalities are to be found in the republic and in the employ of the government.

In the matter of sports, many English games have become popular with the Argentines, among which are football, rowing and wrestling; but it is equally true that sports and amusements of other nations are enjoyed by the people.

It is not the writer's purpose to enter into a lengthy discussion as to the merits of the various views expressed by different ones who have lived in the Argentine, as individual impressions will naturally differ; but the opportunities and conditions existing in a foreign country must be considered from other points of view besides that of engineering to be fairly judged.

E. L. VERVEER.

518 West 111th St., New York City,
Oct. 17, 1914.

The Largest Installations of Cooling Towers, it is claimed, are those being made for the Texas Power Co. at Waco, Tex. There is a steel shell 100 ft. long, 18 ft. wide and 40 ft. high, with six pairs of fans 10 ft. in diameter operating at 250 r.p.m., belt driven by motors located above each pair of fans in small motor houses. The fan-housings are extended from the shell and are provided with doors so that they may be opened to permit unobstructed entrance of air for winter operation under natural draft. There is a platform serving the three motor houses and reached by a ladder from the ground. An upper gallery is built on the level of the water distributor and regulating valves which control the discharge to each compartment of the shell. Any section may be cut off for inspection or cleaning without interfering with the operation of the others. Plans are made for an additional installation of equal capacity at one side of the present outfit, which is designed to cool 600,000 gal. per hour. These towers were made by the Wheeler Condenser & Engineering Co., of Cartaret, N. J., at the direction of Sargent & Lundy, Consulting Engineers, Chicago.

The Twenty-First International Irrigation Congress

BY F. H. NEWELL*

The restoration of confidence in irrigation and the continuation of developments in the arid West were the underlying thoughts at the twenty-first International Irrigation Congress at Calgary, Canada, Oct. 5 to 8.

The confidence which is so essential to further irrigation development must come through achieved success in the work already undertaken. In other words, the settler now on the ground, who is producing crops from the irrigation works already built, must be in a prosperous condition before other settlers and other works can be had. If the average irrigator is contented and prosperous, he will attract new men to the opportunities offered. It must, therefore, be seen that the man on the ground is making good, and when this is done there will be little to fear for the future.

These and similar ideas were developed at the Congress. It was shown that the construction of irrigation works and the reclamation of arid land are still in advance of the problem of settlement. There are now millions of acres of good land provided with water at a cost of upward of fifty dollars an acre which are not being tilled. The settlers on these lands are achieving a certain degree of success, but the great need is for more money, or cheaper money, in order to make it practicable for the settler to utilize his opportunities to the largest degree. He needs more livestock, more machinery, better tilling of the soil. Although the security which he can offer is fairly good, his opportunity to borrow money is limited, and the rates charged him are from 8 to 12%, or even more—enough to destroy any less prosperous business.

It was urged that while the reservoirs should be retained by the Government under careful supervision, the irrigation works themselves should be turned over to the people benefitted at the earliest practicable moment, under some form of organization such as the irrigation district, created under state law and having power of taxation. The present voluntary irrigation associations are found to be largely powerless to enforce the will of the majority, but the irrigation district, with powers comparable to those of a municipality, can and will enforce proper rules, and will deal effectively with present evils, such as those of the "water hog" who is wasting water to the detriment of the community. While the Government may be practically powerless in these matters, local public sentiment, enforced by a strong organization, can reach the desired end.

No new projects should be undertaken until the present schemes initiated under the Carey Act or by private enterprise have been thoroughly investigated. If, as a result of such investigation, some of these schemes appear to be meritorious, effort should be centered on completing them, but with the understanding that no work should be approved, either for Government or private investment, unless it appears that it can bear an interest charge of at least 3%. During the first three or four years, however, no interest should be paid, but an equivalent amount should be added to the construction cost and distributed over annual installments to be paid in the future.

*Director, U. S. Reclamation Service, Washington, D. C.

The majority of large irrigation projects initiated under the provisions of the Carey Act or of irrigation districts so far have not been financially successful. The General Land Office has presented figures showing that of the 3,700,000 acres segregated at the request of various states, under the law of 1894, there have been reclaimed and patented about 438,000 acres, as follows:

State	Acres	State	Acres
Idaho	247,000	Montana	15,000
Oregon	45,000	Wyoming	131,000

In addition to the above, there are pending 384,000 acres, so that there is reasonable probability of patents issuing to about 822,000 acres of lands reclaimed under the terms of this Act. Possibly, other projects may be completed so that we may expect that ultimately there will be reclaimed nearly a million acres. There are 34 projects which are at a standstill, involving 1,500,000 acres, and these may be considered to be wholly, or in part, failures. The difficulties are about as follows:

No. of projects	Conditions	Areas involved, acres
9	Financial troubles	850,000
6	Difficulties unknown	180,000
5	No water	125,000
4	Defective engineering	200,000
3	Defective titles	130,000
3	Accidents after construction	65,000
4	Bad management	70,000

Eleven projects, involving 500,000 acres, have succeeded fairly well.

The irrigation districts organized under state law have had a somewhat similar history, but as a result of the experience gained the laws are being improved, so that now we have every reason to hope that districts can be successfully organized and operated under state law. The advantage which this plan possesses is that the irrigation district has behind it, with possibility of taxation, all of the real estate and other values within the confines of the district. Even if disaster comes, there is possibility of renewing effort through wise exercise of the taxing power.

As the outcome of the discussion of the above mentioned and other topics, the congress was convinced that it still has a mission to perform and continued its permanent organization with a view to meeting probably in California in 1915, future activities to be concentrated largely on the problems of settlement and of securing the prosperity of the irrigator.

A Simplified Solution for Some Statically Indeterminate Problems in Building Construction

By G. A. Maney*

Two properties of the bending-moment curve for beams are used, in this article, to simplify the solution of all problems involving statically indeterminate bending moments. These properties are:

(1) The area under the M/EI curve, between any two points on the elastic curve, measures the change of the slope of the elastic curve between these points. M, E and I represent the bending moment, the modulus of elasticity of the material and the moment of inertia of the beam section, respectively.

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(2) The statical moment of the area under the M/EI curve between any two points on the elastic curve, about one of these points, measures the deflection at this point from the tangent to the elastic curve at the other point.

These properties have long been well known among authorities on the subject of mechanics, but their great possibilities for indeterminate problems have been neglected. Their simplicity and ease of application to such problems, as compared with applications of the principle of least work and of the differential equation of the elastic curve (generally used at the present time), makes this method of solution a desirable one.

By the use of the two properties mentioned, we are enabled to write directly, from simple inspection of the M/EI curve, as many simple linear equations between the bending moments as are necessary for a complete solution.

The method here described has been used to advantage by the Illinois Experiment Station. It simplified the analysis of the results of tests of reinforced-concrete construction with reference to combined slab (beam) and column action—in resisting bending moments due to loads applied on only one panel, or eccentric loading.

Two specific problems will be worked through by this method to demonstrate its ease and simplicity of application.

A proof of the above-cited properties, which are probably not very widely recognized among engineers, will be given, after which the problem of the elevated slab for the Soo Line Chicago freight terminal will be solved.

An excellent illustration of the application of these properties is found in the extremely short and direct derivation of the theorem of three moments, given at the end of this article.

Proof of Properties

Let

M = bending moment;
I = moment of inertia of the sectional area of the beam;
E = modulus of elasticity of the material;
c = distance from the neutral axis to the extreme fiber;
dS = element of length along the neutral axis;
$d\theta$ = increment of slope in the distance dS, due to bending.

I. At any point on the extreme fiber of a beam, Fig. 1, we have the unit-stress Mc/I, and the corresponding stretch or compression in length dS is, therefore,

$$\frac{Mc}{EI}\,dS$$

We obtain the change of slope in the distance dS by dividing the deformation for a distance dS along the extreme fiber by c, the distance from the extreme fiber to the neutral axis.

The slope increment in the distance dS is, therefore,

$$d\theta = \frac{M}{EI}\,dS$$

A glance at the M/EI curve, Fig. 2, makes it evident that the quantity $(M/EI)\,dS$ equals the area under this curve for a distance dS along it. The sum of the changes of angle in lengths dS equals the total change of

the slope; in other words, the sum of all the elementary areas $(M/EI)\ dS$ between any two points of the beam equals the area of the (M/EI) curve between these two points. Therefore, the area under the M/EI curve between any two points is an exact measure of the change in angle between the two tangents to the elastic curve at these points, due to bending.

II. Suppose we desire the deflection of the point N' from the tangent MN.

We will assume here that the displacements of the neutral axis due to bending are so small that the projection of the neutral axis after bending upon the neutral axis before bending has practically the same length as the neutral axis before bending. This is the assumption commonly made in deriving the differential equation of the elastic curve.

Let S, Fig. 1, be the distance from the middle of ele-

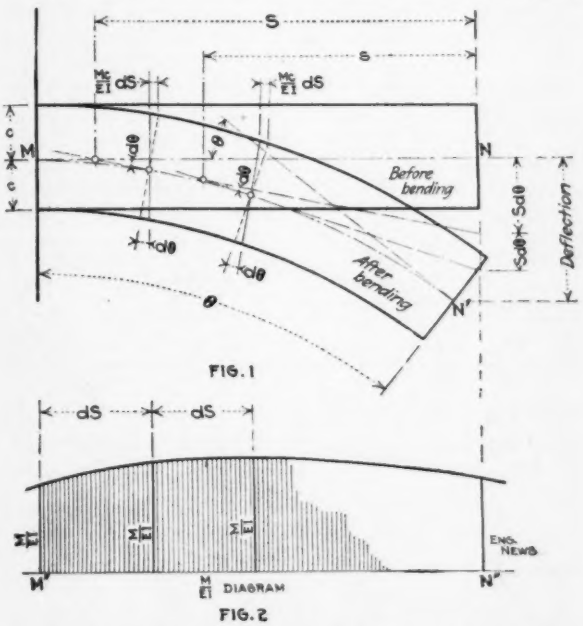

FIG. 1

FIG. 2

FIG. 1. PORTION OF BEAM, ORIGINAL AND BENT POSITIONS

FIG. 2. MOMENT DIAGRAM OR $\frac{M}{EI}$ DIAGRAM

mentary portion dS to the point N whose deflection is desired. The effect of the elastic curvature $d\theta$ of the portion dS upon the deflection at N is measured by the product $S\,d\theta$. But $d\theta$, as previously proved, is equal to the area of the $\frac{M}{EI}$ diagram corresponding to the portion dS; therefore, the product $S\,d\theta$ is the statical moment of this area about the point N'' corresponding to N, whose deflection is desired. It follows that the total deflection at N, being due to the bending in all the elementary portions dS, is equal to the statical moment of the entire M/EI curve about point N''.

APPLICATION TO ONE-STORY BENT, SOO TERMINAL

In load tests of the reinforced-concrete Soo line freight terminal at Chicago, made under the direction of Prof. A. N. Talbot, a question of statically indeterminate bending moments came up. An attempt was made to analyze the action of the structure with one panel loaded and all the adjacent panels empty. The object of the analysis was to determine what effect the relative moments of inertia and lengths of the sections of the floor and the columns would have upon the bending moments.

The conditions which were assumed as being approximately true in one of the analyses made are represented in Fig. 3. For the heavy elevated-track floor slab a condition of approximate fixity was assumed for the outside edges of the panels surrounding the loaded area, and also for the bottom or base of the columns. The column and slab moments were to be determined.

The problem was first solved by the method of least work, and values for the various moments were obtained which checked themselves in all the limiting cases. Then the method here described was used, and an exact check was obtained for every value. The amount of detailed analysis necessitated was only about 10% of that required by the analysis based on the principle of least work.

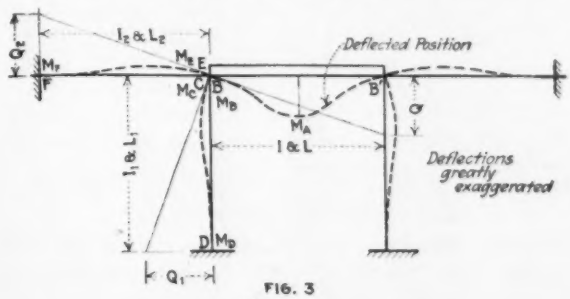

FIG. 3

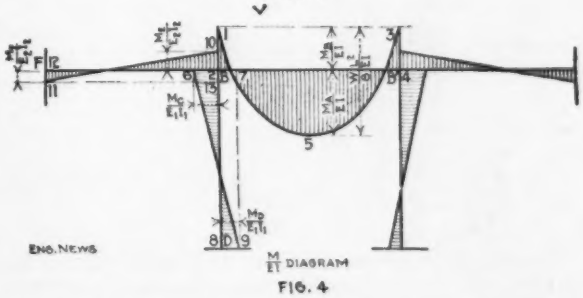

FIG. 4

FIG. 3. ONE-STORY BENT ANALYZED FOR SOO TERMINAL TESTS

FIG. 4. MOMENT DIAGRAM FOR SOO BENT

As sketched in Fig. 3, there are six unknown bending moments, M_A, M_B, M_C, M_D, M_E and M_F. Between these six unknowns, we can only set up two equations from the conditions of statics. These are

$$M_B = M_E + M_C \qquad (1)$$

$$M_A + M_B = \tfrac{1}{8}\, w\, L^2 \qquad (2)$$

The remaining four equations can be written with the aid of the two moment-curve principles above demonstrated. The distribution of the M/EI values is indicated by the curves of the diagram Fig. 4.

The condition of rigidity of the joint at B means that the two tangents at this point are always perpendicular to each other. From this condition we get two independent equations:

$$\frac{Q}{L} = \frac{Q_2}{L_2}, \text{ and } \frac{Q_1}{L_1} = \frac{Q_2}{L_2}$$

Here Q, Q_1 and Q_2 are the deflections of the points B', D and F. The deflection Q is measured by the statical moment of the area 1-5-3-4-2, Fig. 4, about B', or the

moment of the area 1-5-3 minus the moment of the area 1-2-4-3.

Therefore

$$Q = \frac{\frac{1}{8} w L^3 \times \frac{2}{3} L \times \frac{1}{2} L - M_B \times \frac{1}{2} L \times L}{EI}$$

The deflection Q_2 is measured by the statical moment of the area 12-2-10-11 about the point F, equal to the moment of the area 11-13-10 minus the moment of the area 12-11-13-2.

Therefore

$$Q_2 = \frac{(M_E + M_F) \frac{1}{2} L_2 \times \frac{2}{3} L_2 - M_F \times L_2 \times \frac{1}{2} L_2}{E_2 I_2}$$

And knowing that Q/L equals Q_2/L_2 we get

$$\frac{\frac{w L^3}{3 \times 8} - \frac{M_B L}{2}}{EI} = \frac{\frac{(M_E + M_F) L_2}{3} - \frac{M_F L_2}{2}}{E_2 I_2}$$

Also since Q_1/L_1 equals Q_2/L_2 we get

$$\frac{\frac{(M_C + M_D) L_1}{3} - \frac{M_D L_1}{2}}{E_1 I_1} = \frac{\frac{(M_E + M_F) L_2}{3} - \frac{M_F L_2}{2}}{E_2 I_2}$$

The deflection of the point B from the tangent to the elastic curve at D is zero since the point B remains unchanged and the tangent at D is fixed by the conditions of the problem. Therefore, the statical moment of the area 8-2-6-9 about the point B is zero, or, what is the same, the moment of the area 6-9-7 about the point B minus the moment of the area 2-7-9-8 about the point B is equal to zero. From this we get

$$\frac{(M_C + M_D)\left(\frac{L_1}{2}\right)\left(\frac{L_1}{3}\right) - M_D L_1 \left(\frac{L_1}{2}\right)}{E_1 I_1} = 0$$

or

$$M_C = 2 M_D$$

The same thing being true with regard to the tangent at the point F we may immediately write

$$M_E = 2 M_F$$

The relations indicated in these two equations may be also derived by a simple integration of the elastic curve for these end conditions.

Assuming that the modulus of elasticity is the same in all cases and simplifying we get:

$$M_B = M_C + M_E \tag{1}$$

$$M_A + M_B = \frac{w L^2}{8} \tag{2}$$

$$\left(\frac{w L^2}{4} - 3 M_B\right)\frac{L}{I} = (2 M_E - M_F)\frac{L_2}{I_2}$$

or

$$\frac{w L^2}{4} - 3 M_B = K_1 (2 M_E - M_F) \tag{3}$$

where

$$K_1 = \frac{L_2 I}{L I_2}$$

$$2 M_C - M_D = K_2 (2 M_E - M_F) \tag{4}$$

where

$$K_2 = \frac{L_2 I_1}{L_1 I_2}$$

$$M_C = 2 M_D \tag{5}$$

$$M_E = 2 M_F \tag{6}$$

Substituting (5) and (6) in (4) we get,

$$M_C = K_2 M_E \tag{a}$$

Substituting (a) in (1) we get,

$$M_B = (1 + K_2) M_E \tag{b}$$

Substituting (b) and (6) in (3) we get,

$$\frac{w L^2}{4} - 3 (1 + K_2) M_E = \frac{3 K_1}{2} M_E$$

Therefore

$$M_E = \frac{w L^2}{12}\left(\frac{2}{K_1 + 2 K_2 + 2}\right)$$

$$= \frac{w L^2}{12}\left(\frac{2 L L_1 I_2}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}\right)$$

$$\therefore M_E = C \frac{w L^2}{12}$$

and

$$C = \frac{2 L L_1 I_2}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}$$

From (6)

$$M_F = \frac{C}{2}\left(\frac{w L^2}{12}\right) = \frac{w L^2}{12}\left(\frac{L L_1 I_2}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}\right)$$

From (a)

$$M_C = C K_2 \left(\frac{w L^2}{12}\right) = \frac{w L^2}{12}\left(\frac{2 L L_2 I_1}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}\right)$$

From (5)

$$M_D = \frac{C K_2}{2}\left(\frac{w L^2}{12}\right) = \frac{w L^2}{12}\left(\frac{L L_2 I_1}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}\right)$$

From (1)

$$M_B = C (1 + K_2)\frac{w L^2}{12}$$

$$= \frac{w L^2}{12}\left(\frac{2 L L_1 I_2 + 2 L L_2 I_1}{L_1 L_2 I + 2 L L_2 I_1 + 2 L L_1 I_2}\right)$$

From (2)

$$M_A = \frac{w L^2}{8} - M_B$$

It is an easy matter to test some of these values for the limiting cases. Take for instance the value for M_B. There are four conditions of the members BD and BF which would produce a condition of fixed ends for the central span BB': If I_1 or I_2 become very large in comparison with the remaining values of I, or if L_1 or L_2 become very short in comparison with the remaining values of L, we will get a condition approaching fixed ends for the central span. In all four of these cases it is seen that the value of M_B approaches $wL^2/12$, which is the value of the end moment for a fixed span with a uniform load. When these above mentioned values of I and L approach the other extreme we have a condition of a freely supported simple span, where M_B approaches zero, as a substitution will show.

When I_1 equals zero, I equals I_2, and L equals L_2, we have the condition of a beam of uniform section with three equal spans, fixed at the two end supports and supported freely on the two intermediate supports. Here again the values of M_B and M_E check themselves by becoming $wL^2/18$, while M_F becomes $wL^2/36$. For this last case see Fig. 5.

The Theorem of Three Moments

From a theoretical point of view it is interesting to find how simple a derivation of the theorem of three moments becomes when the moment-area method is applied.

Take the case where the two adjacent spans have different lengths and different uniform loads, and the beam section for each span has a different moment of inertia, as indicated in Fig. 6. By the use of the theorem of three moments we obtain $n - 2$ equations between n unknown

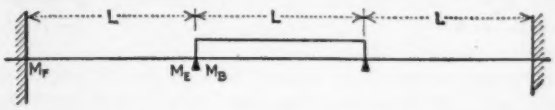

Fig. 5. A Limiting Case for the One-Story-Bent Formulas. ($I_1 = 0$)

moments at n supports. The remaining two equations can be written from a knowledge of the degree of fixity of the two end supports.

For each support except the two end ones we have a condition which determines the relation between the moment at the support considered and the moments at the two adjoining supports. The determining condition is that of continuity at the support considered, which means that the two adjacent spans have a common tangent at this support.

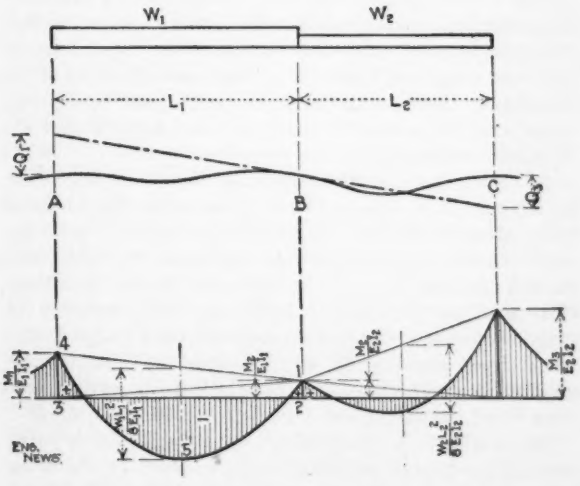

Fig. 6. Portion of Continuous Beam, and Moment Diagram

Let Q_1 and Q_3 be the deflections at A and C, Fig. 6, from the common tangent to the beam at B, the middle support of the three. Let L_1, w_1, I_1 be the lengths, load per lineal foot, and moment of inertia of beam cross-section in the left-hand span, and L_2, w_2, I_2, the corresponding quantities for the right-hand span.

The deflection Q_1 is measured by the statical moment of the M/EI area 3-4-5-1-2 about support A. This again is equal to the moment of the area 4-5-1 minus the moment of the triangle 1-2-3 minus the moment of the triangle 1-3-4, so that

$$Q_1 = \frac{w_1 L_1^3}{8 E_1 I_1}\left(\frac{2 L_1}{3}\right)\left(\frac{L_1}{2}\right) - \frac{M_1}{E_1 I_1}\left(\frac{L_1}{2}\right)\left(\frac{L_1}{3}\right) - \frac{M_2}{E_1 I_1}\left(\frac{L_1}{2}\right)\left(\frac{2 L_1}{3}\right)$$

Similarly,

$$Q_3 = \frac{w_2 L_2^3}{8 E_2 I_2}\left(\frac{2 L_2}{3}\right)\left(\frac{L_2}{2}\right) - \frac{M_3}{E_2 I_2}\left(\frac{L_2}{2}\right)\left(\frac{L_2}{3}\right) - \frac{M_2}{E_2 I_2}\left(\frac{L_2}{2}\right)\left(\frac{2 L_2}{3}\right)$$

Since $Q_1/L_1 = -Q_3/L_2$ on account of the common tangent over the middle support, we can immediately write the theorem of three moments by substituting and reducing. Giving the moments at the support the minus sign we finally get

$$\frac{w_1 L_1^3}{24 E_1 I_1} + \frac{w_2 L_2^3}{24 E_2 I_2} + \frac{M_1 L_1}{6 E_1 I_1} + \frac{M_2}{3}\left(\frac{L_1}{E_1 I_1} + \frac{L_2}{E_2 I_2}\right) + \frac{M_3 L_2}{6 E_2 I_2} = 0$$

Now, assuming that $E_1 = E_2$ and $I_1 = I_2$, we get the familiar form

$$M_1 L_1 + 2 M_2 (L_1 + L_2) + M_3 L_2 = -\left(\frac{w_1 L_1^3}{4} + \frac{w_2 L_2^3}{4}\right)$$

Annual Convention of the American Electric Railway Association

Th 33d annual convention of the American Electric Railway Association and meetings of the allied bodies of engineers, traffic men, accountants, and claim agents were held in Atlantic City, N. J., Oct. 12 to 16. There was the usual huge exhibition of the affiliated manufacturers and supply dealers; in this were prominent novel developments in track-signal systems, automatic "flagmen," illuminated car signs, vitreous-enamel interior fittings, ventilated and pressed-steel motors, headlights, etc. There was perhaps a falling off in the number of cars shown, in the amount of overhead and track work, in wheels and trucks, and in construction, shop and power-plant apparatus—though the amount of equipment, large and small, was not meager.

The very satisfactory plan was followed of having parallel sessions of the special associations in the mornings and meetings of the present body in the afternoon for consideration of the matters of general interest. In addition to the serious work noted below, an extensive entertainment program was arranged to fill the spare hours of all in attendance. The registration was about 2400—a falling off of some 20% generally attributed to the demands for economy throughout the industry.

American Association

Addresses—Under the title, "Mooted Principles in Valuation," F. W. Stevens, Chairman Valuation Committee New York Central Lines, discussed a single matter—the meaning of "value." It appeared to him very plain in economics and in law that the word had but one true use. This covered the worth of anything, expressed in money, to a willing purchaser. He discussed the rate cases of the U. S. Supreme Court to support his opinion (1) that there was only this one worth and that it applied alike in ratemaking, taxation and condemnation; (2) that it was immaterial whether property was paid for by money from stocks and bonds, or by excess earn-

ings, or was donated; (3) that all property had to be protected alike under the constitutional guarantees and therefore appreciation in worth had to be included; (4) that reproduction cost was not value, though often a means of approach.

Mr. H. C. Donecker, Public Service Ry., Newark, N. J., in a paper "Organization and Functions of a Valuation Staff," quoted extensively from the writings of experienced appraisers to show the cost of appraisals, the methods of procedure, the qualifications of the staff and the use of the valuation results. He recited arguments for having the valuation carried on by company forces, for making it convincing, and for the economy of a committee in charge with divided responsibility.

In the discussion, Philander Betts, Chief Engineer New Jersey Board of Utility Commissioners, advanced the opinion that the purpose of the valuation should affect the make-up of the staff. He favored having the inventory made by a joint force of company and commission employees. Henry Floy, Consulting Engineer, New York, commended Mr. Betts' idea, but called attention to the unfitness of the ordinary construction or operating engineer to prepare a valuation for court or commission scrutiny. He criticized an existing lack of that full coöperation of companies and commissions which was needed for best results.

In "Aspects of Workmen's Compensation Laws," H. A. Bullock, Brooklyn, N. Y., briefly reviewed the economic, social and political conditions which made compensation laws a live issue in so many states. The haste in which the laws were being prepared and the disagreement among expert advisors tended to result in provisions burdensome on both employee and employer; the electric railway industry was advised not to stand by and let unfair sections go to unchallenged enactment.

In the paper, "Electric Railway Securities from the Company Standpoint," Calvert Townley, of New York, called on railway men to stop apologizing for early sins which, he claimed, the public had condoned and in spite of which the public had profited from the utilities as much as the promoters. The large sums needed to develop traction lines to meet the demands made on them could not come from earnings and would come from private investors only when adequate return could be secured—all of which he claimed should be more evident in the exercise of public control.

"A Zone System of Fares," by R. B. Stearns, of Milwaukee, described the system worked out for Milwaukee to apportion fares equitably according to length of ride. Data on zone operation abroad was also presented. It was voted to reprint the paper in pamphlet form for distribution.

Reports—The address of the President, C. N. Black, of San Francisco, was a review of the year's work and a demand for larger rates of return on capital. The annual report of the Secretary-Treasurer, E. B. Burritt, New York, showed that the 379 member companies now represented 36,000 of the 43,000 miles of electric-railway track and 500 million of the 533 million dollars revenues received. There are 2898 individual members. Some $76,000 income was received and about $71,000 spent.

The Committee on Education, Chairman, H. H. Norris, New York, reported that it had canvassed the member companies, and had found practically none having apprentice systems with regular courses of training. No instruction of a school nature was found, little coöperation with outside schools, and no system of encouraging educational progress in employees. However, a favorable attitude was noted toward a correspondence-school course under the auspices of the Association. Such courses, arranged for mechanical-shop, electrical-shop, line, track, power-house and substation men were outlined.

The Electrolysis Committee, Chairman, Calvert Townley, New York, reported the formation of a joint national committee on electrolysis and advised discontinuing independent study.

The Committeee on Insurance and Fire Protection, Chairman, H. J. Davies, Cleveland, gave statistics of the member companies' experience during the year. The fire losses were the highest of any year but one, and the rate the lowest; the average rate of 56c., it was held, could not be decreased unless present hazards were reduced.

The Joint Committee on Joint Use of Poles, Chairman, W. J. Harvie, of Syracuse, presented revised specifications, which had been worked out by the several interests involved, together with a model agreement. On account of protests that various practices were not universally applicable and, if broadly recommended, would constitute evidence of negligence when not followed, the convention received the report merely as guiding information.

The Committees on Ways and Means and Changes in Constitution recommended raising the company dues from the present range of $15 to $600 (depending on gross earnings) to $25-$750 so that the activities of the association need not be curtailed. This plan is expected to increase the annual revenues by some $16,000 and was unanimously accepted.

A Committee on Compensation for Carrying Mail, Chairman, M. C. Brush, Boston, described the consideration given by the Senate Committee revising mail pay and showed high hopes of fair treatment if proper facts could be presented. A Committee on Federal Relations, Chairman, A. W. Brady, Anderson, Ind., reported efforts to have electric roads recognized as a distinct class from steam roads and, on account of local regulation, exempt from provisions of drastic bills requiring steel cars, block signals, etc.

The work of the new Bureau of Fare Research in securing uptodate information was presented by the Committee on Cost of Passenger Transportation, Chairman, J. D. Mortimer, New York.

The Public Relations Committee, Chairman, T. N. McCarter, Newark, N. J., transmitted the report of a subcommittee which recommended the establishment of a bureau of public relations to disseminate reliable information and to coöperate with similar bureaus. The report also presented a plan of publicity through Chautauqua lectures, technical educators, technical, civic, and economic societies, popular magazines, and newspaper advertising. Eventually, some $50,000 per year was to be sought, but for the present, a start would be made by asking certain leaders of the industry to deliver addresses. A code of utility principles was enunciated much as follows: (1) Adequate service is the first obligation; ample fares are necessary to provide it and yield a fair return; (2) regulated private ownership is preferable to government ownership or operation; (3) local transportation should be a monopoly; (4) short-term

franchises discourage good service and check growth; (5) capitalization should be authorized on terms which will produce needed funds; and securities previously issued in accordance with law should be valid obligations; (7) adequate wages should be recognized but utilities should be protected against strikes and excessive demands; (8) holding companies are economically sound; (9) in appraisal, all methods of valuation should have due consideration; (10) full and frank publicity should furnish public and investor with proper information.

Engineering Association

Reports—After many routine and organization matters, various technical reports were presented. The Power Distribution Committee submitted specifications for rubber-insulated wire and cable, for round and grooved copper trolley wire, for high-voltage paper-insulated lead-covered cables, for 600-volt overhead construction and for galvanizing iron and steel. In an appendix, the design of reinforced-concrete poles was briefly outlined.

A Board of Accident Prevention described how greater safety was actually being secured by improved equipment and special devices, by education of general public, of school children and employees, by traffic regulation, etc. A paper on "Accident Prevention," by F. C. Henderschott, New York, gave statistics of the improvement secured by a few of the thousand odd concerns which have instituted safety systems.

The Block Signal Committee, Chairman, J. M. Waldron, New York, supplemented its 1913 report with a new digest of state laws, a bibliography, descriptions of light signals for interurban roads, a summary of new installations, standard rules for operation of trolley-contact signals, a summary of contemplated signal installations, a description of a new automatic train stop (Horne-Crane type), and a paper by Gaylord Thompson on "Braking Distances." A standard spectacle and a standard clearance diagram were offered; the former was accepted by the meeting but the latter was returned for further study in connection with other committees. The operating rules presented, but slightly modified, were accepted as standards. The recommendation was adopted that signal lenses of not less than 8⅜ in. and 5⅜ in. diameter, respectively, should be considered good practice for high-speed and moderate-speed operation.

A Committee on Transportation Engineering gave figures for a single-phase line in the Middle West where trains had secured 158% increase in seating capacity and 122% in ton miles with 76% increase in power consumption. A tendency was reported in freight and express service to use a motor car or locomotive with trailers instead of multiple-unit trains. Several favorable car designs were shown.

The Equipment Committee, Chairman, F. R. Phillips, Pittsburgh, recommended further investigation of the standard specifications for heat-treated axles, of possible standard specifications for gears and pinions, troubles with M.C.B. journal brasses, and car wiring in relation to fire-protection. Standard specifications were presented and adopted for solid wrought-steel wheels; specifications of "recommended practice" were accepted for air-brake hose. As an appendix, a paper by L. M. Clark, of Indianapolis, outlined the existing common faults of car-lighting and the proper use of reflectors and lamps for improvement. Data from a large number of tests were given for guidance in fixing size and character of new equipment.

The Buildings and Structures Committee, Chairman, C. F. Bedwell, Newark, N. J., reported approval by the National Fire Protection Association of last year's protection rules for electric-railway properties. General specifications and a form of contract for building construction were presented for consideration as eventual standards. A synopsis of power-house design was submitted for criticism as a part of proposed study of modern practice.

The Committee on Engineering Accounting, Chairman, J. P. Barnes, Brooklyn, N. Y., showed the need of greater attention to applying overhead interdepartmental charges to cost figures of construction that there be no self deception and false economy. A scheme of account analysis was presented so that engineering and executive departments could compare in detail, the original estimates and final costs, etc.

The report of the Power Generation Committee, Chairman, B. F. Wood, New York, was a collection of papers on (1) status of machine stokers (in which it was held that the field was narrowing down to chain grates for high-ash high-volatile fuels and the underfeed type for other coals); (2) use of special reactance coils to protect power apparatus and feeder circuits; (3) advantages and disadvantages in the use of 60-cycle rotary converters.

At some of the sessions of other allied groups, a few papers of more or less engineering interest were presented; one was particularly notable—"The Accounting Treatment of Depreciation," by Robert Sealy, New York. In this it was urged that many of the difficulties in comprehending the subject would be overcome if so called "depreciation" be treated as a replacement insurance. An insurance scheme of computing annual depreciation contributions was offered, but with the remark that life limits due to wear, accident, obsolescence, inadequacy and public demand were all so uncertain that close computations of annual amounts for reserves were useless.

The Committee on Way Matters, Chairman, C. S. Kimball, Washington, D. C., discussed track foundations for paved streets, T-rails and alloy-steel rails. In regard to foundations, it was stated that permanency was the main object. Many soils considered unfit as a foundation, without concrete, could be made serviceable by drainage. Good foundation drainage hindered stray currents and electrolysis. Practice of rolling subgrade and ballast was increasing and was recommended. Current practice seemed to be away from solid concrete construction. Four types of foundation were recommended. The advantages and disadvantages of T-rail construction in streets were recited and conclusions given from data secured that (1) diverse opinions still excited, (2) granite or brick headers are generally used along gage lines and under heads, (3) portland-cement or asphaltic-cement grout is best for filling the space next to rails, (4) tendency of vehicles to follow rails is about the same with all rails. It was recommended that T-rails in permanent pavements should be 7 in. high. The tonnage of alloy-steel rails rolled continued to decrease; the production of bessemer rails also decreased, but use of open-hearth steel correspondingly increased.

The Heavy Electric Traction Committee, Chairman, E. R. Hill, New York, presented a revised diagram of recom-

mended clearances for overhead conductors, showing 15 ft. 3 in. for direct-current rails, and advised continuing the study of third-rail clearances and electric locomotives.

NEW OFFICERS

The following officers were elected for the parent association: President, C. Loomis Allen, President of the Newport News & Hampton Ry., Gas & Electric Co., Syracuse, N. Y.; Vice-presidents, C. L. Henry, President of the Indianapolis & Cincinnati Traction Co., Indianapolis; J. A. Beeler, Vice-president of the Denver Tramway Co., Denver; L. S. Storrs, President of the Connecticut Co., New Haven; T. S. Williams, President of the Brooklyn Rapid Transit Co., New York.

The Engineering Association elected the following: President, L. P. Crecelius, Electrical Engineer of the Cleveland Ry.; Vice-presidents, John Lindall, Superintendent of Rolling Stock and Shops, Boston Elevated Ry.; B. F. Wood, Chief Engineer, United Gas & Engineering Co., New York; F. R. Phillips, Superintendent of Equipment, Pittsburgh Rys.

The new presidents of the other associations are: M. C. Brush, second Vice-president of the Boston Elevated Ry. (Transportation and Traffic); C. S. Mitchell, Auditor of the Pittsburgh Rys. (Accountants); William Tichenor, Claim Agent of the Terre Haute, Indianapolis & Eastern Traction Co. (Claim Agents); E. H. Baker, Second Vice-president of the Galena Signal Oil. Co., New York (Manufacturers.). The Secretary-Treasurer is appointed and not elected. The present incumbent, E. B. Burritt, continues in office and serves in similar capacity for the allied associations, except that M. R. Boylan, Public Service Ry., Newark, N. J., serves for the accountants and H. G. McConnaughy, Dearborn Chemical Co., New York, for the manufacturers.

Should Members of the Engineering Profession in Pennsylvania Be Subject to State Examination and License?

The Pennsylvania legislature a year ago created a Commission of engineers which was instructed "to investigate the advisability of licensing engineers, within this Commonwealth, who are engaged upon the construction, maintenance and operation of works, public or private, within this Commonwealth, where unusual hazards are offered to the lives, health and property of the citizens of this Commonwealth; and to recommend to the Governor such measures as, in their judgment, should be enacted into laws."

Governor Tener appointed as this Commission John Price Jackson, Samuel A. Taylor, Geo. S. Webster, F. Herbert Snow and J. Murray Africa. The Commission held public hearings at Pittsburgh on Oct. 15 and 16, at Philadelphia on Oct. 19 and will hold a further hearing at Harrisburg on Oct. 28 at the headquarters of the Engineers Society of Pennsylvania, which has called a special meeting for that date.

In order to stimulate discussion, the Commission has printed a draft of a bill which would require engineers engaged in any sort of professional engineering work, whether civil, mechanical, mining, electrical or marine, to be registered as professional engineers by a State Examining Board, the practice of engineering of any sort without such registration being made a misdemeanor, punishable by fine and imprisonment.

The hearings at Pittsburgh and at Philadelphia were well attended by leading men in the profession. The sentiment expressed was well nigh unanimous against the proposed bill and in most cases the opinions were adverse to any state interference whatever with the practice of engineering.

A number of those attending the sessions represented engineering societies in Pennsylvania or engineering schools in the state. The Philadelphia Association of Members of the American Society of Civil Engineers held a special meeting at which the proposed legislation was discussed at great length; the final vote was nearly unanimous against the proposed bill.

A committee was appointed to represent the Association before the Commission and prepared a report which is so admirable a summary of the principal arguments against engineers license legislation that we print it in full below.

The State Commission, after conclusion of the hearings, will formulate its findings in a preliminary report to the Governor by Nov. 1 and its final report before the end of the year. The Commission is pursuing a judicial attitude in its inquiry and by its questions of those who appeared before it appeared anxious to obtain constructive criticism as well as destructive criticism. In other words, the Commission appeared anxious to ascertain not merely what were the defects and dangers of the particular bill offered for discussion, but what legislation, if any, is desirable in connection with the practice of engineering to safeguard the public welfare.

The Committee's report above referred to is as follows:

ARGUMENTS IN OPPOSITION TO THE PROPOSED ACT FOR LICENSING PROFESSIONAL ENGINEERS IN THE STATE OF PENNSYLVANIA

The Committee appointed to advance arguments against the proposed Act for Licensing professional engineers in the State of Pennsylvania desires, first, to present certain general considerations, and then to treat the subject under the following headings:

A Objections to any plan looking to the licensing of engineers in the State of Pennsylvania.

B Specific objections to the terms of the particular Act proposed by the Engineers' Commission.

C A proposed plan for accomplishing the avowed purposes of this Act in a more direct and effective manner.

GENERAL CONSIDERATIONS

1 In support of plans for licensing engineers in a given state, the claim is usually made that there is the same need of licensing engineers as of licensing lawyers and physicians, whereas the conditions in engineering are, in fact, wholly different from those in law and medicine. In the latter professions the public interests cannot be measurably protected against charlatans and quacks except through licensing on the basis of individual examinations, although experience has shown that even that safeguard is inadequate for the purpose stated. It may, in fact, be confidently affirmed that no plan of licensing can serve to eliminate unscrupulous, though technically competent men, who will not hesitate to resort to unprofessional practices in so far as that may be possible with safety to themselves.

In engineering, unlike law and medicine, the public may be protected against incompetent or unscrupulous practice, not by the licensing of engineers, but by state approval of plans before execution, by state inspection during construction, and by other means to be set forth in more detail hereafter.

2 The fact that the licensing of lawyers and physicians is required in every state of the Union, whereas provisions for the licensing of engineers is made in only two states (Louisiana and Wyoming) indicates that the fundamental differences, from the standpoint of public interests, between the professions of law and medicine on the one hand, and that of

engineering on the other, are widely recognized. The burden of proof in justification of an Act designed to license engineers in the State of Pennsylvania rests, therefore, upon the sponsors of such an Act.

3 In support of a plan for licensing professional engineers it may be claimed that it is necessary for the promotion of safety and the protection of life, health and property. It will be shown, however, that these ends cannot be attained through such an Act, and that other measures would be far more effective toward their accomplishment.

A Objections to any Plan Looking to the Licensing of Engineers in the State of Pennsylvania

I. The licensing of independently established consulting engineers is objectionable on the following grounds:

(a) That the plan would afford no reliable guarantee that those who pass prescribed examinations will not design unsafe work through incompetency, carelessness, or under pressure of economical restrictions.

(b) It is a well known fact that the test of examination becomes increasingly inadequate as the value to be set upon experience, and sound judgment based on such experience, becomes increasingly important. It is easy, for example, to determine by examination a candidate's qualifications for such minor positions as rodman, draftsman, etc., but difficult to ascertain by such a test a candidate's fitness for an engineering position involving large and varied responsibilities.

It is impossible to carry in one's head the many data which a seasoned engineer would readily command in his normal office environment, where he has access to reference books, filing cases, personal notes, etc., which would readily enable him to find the desired data; whereas such accessories would be of comparatively little value to an inexperienced man. On the other hand, it is easily conceivable that the latter would outstrip the seasoned man in an examination, since it is almost impossible, in an examination, to set the proper value upon ability and experience as distinguished from mere book knowledge.

(c) The field of engineering is so highly specialized, and the various specialties overlap to such a degree, that no individual can reasonably be expected to qualify in a full sense in more than a single specialty and, in a modified sense, in closely related specialties. The term "professional engineer" is therefore meaningless, except in an inclusive way as covering many specialized titles. Such a speciality as railroad engineering, for example, is properly sub-classified under "Design, Construction, Maintenance and Operation." A given individual may deservedly have attained to the highest rank in Construction, although he may command little or no knowledge of Designing. A fair examination in such a case should be limited to Construction; or better still, the personal record of the individual concerned should be accepted as a far better criterion of his qualifications than any form of examination.

(d) That the licensing of engineers will not serve to avert disaster through improper design or construction is attested by the calamitous failures that have at times attended engineering operations under the direction of engineers deservedly recognized as among the ablest members of the profession—men who would unquestionably have qualified under a proper examination.

II. The licensing of salaried engineers, as distinguished from independent consulting engineers, is objectionable on the following grounds:

(a) The great majority of engineers (probably not less than 90%) hold salaried positions, and the legal and moral responsibility for their professional work should fall on the employing municipalities, corporations or firms, by whom their competency can also be far better determined than through the operation of any legislative Act. The licensing of such engineers would therefore serve no useful purpose, and would be an unnecessary source of expense and annoyance to those affected.

(b) No scheme of examination that can be devised is as effective for determining the degree of professional competency as the principles ordinarily governing the employment, promotion and discharge of salaried men.

(c) Every salaried engineer upon merited promotion to a position of a certain higher responsibility would have to pass an examination prescribed by the state. His rejection upon such an examination would work an undeserved professional injury to the individual concerned, and would justly tend to bring ridicule upon the examining system.

(d) In the promotion of engineers to positions of higher responsibility by corporations whose activities extend beyond the state, such as railroad companies, public-service corporations, etc., the transfer of an individual from another state to Pennsylvania could not take place without subjecting him to an examination involving the contingencies just mentioned.

B Specific Objections to the Terms of the Particular Act Proposed by the Engineers' Commission

I The indefiniteness of certain important features in the proposed Act.

In Section 1, Paragraph 1, the Act is made applicable to those charged with "the designing or general supervision of the operation and maintenance to a greater or less extent of public or private works, etc." The term "greater or less extent" is so indefinite that it may well be asked whether such officers, for example, as railroad supervisors would come under the provisions of the Act.

On the other hand, according to Section 20, Paragraph 2, one of the classes under "Limitation and Scope of Act" is described as "any professional engineer employed as an assistant to a registered professional engineer." This may fairly be assumed to mean that the chief engineer of a railroad company or the chief of a municipal bureau would be required to register as a "Professional Engineer," whereas the innumerable engineers of subordinate rank would be exempt. It is evident that an engineer of subordinate rank on an engineering operation of first magnitude may be charged with much heavier responsibilities than the chief engineer of an engineering operation of minor importance. Again, engineers in subordinate positions on important engineering works may command specialized knowledge not possessed by the chief engineer.

II Certain features of the proposed Act are apparently contradictory.

Thus, in Section 3, Paragraph 3, it is stated among the requirements for eligibility to examinations that the candidate must have "had charge of engineering work as principal or assistant for at least one year." whereas by Section 1, Paragraph 1, only "Professional Engineers" can be "in charge" of engineering work. If these terms were enforced it would apparently mean that an engineer in order to qualify for an examination as "Professional Engineer" in Pennsylvania must have had charge of engineering work in another state for at least one year.

III The proposed Act is incomplete in failing to provide for certain important features, as, for example, the following:

(a) The professional complexion of the examining board. Modern engineering is so highly specialized that it may be confidently affirmed that an examining board consisting of only nine members, no matter how otherwise constituted, would be incompetent to supervise examinations in every important engineering speciality.

(b) Since no individual can possibly qualify as a "Professional Engineer" in the inclusive sense, the act should indicate by title, and define as to scope, the various divisions of professional engineering in which candidates may qualify by examination. The difficulty of doing this is manifest, and this only serves to indicate the futility of determining the competency of engineers for high professional services through examination.

(c) The progress in engineering, especially in certain lines, is so rapid that an engineer who may readily pass a fair examination at a given period, may be utterly unable to qualify under an uptodate examination five or ten years later, unless he has kept well abreast of progress. Therefore, if examinations are to be applied as a test of competency, they should be reapplied at suitable intervals, as is the practice, for example, in the U. S. Army and Navy.

(d) It should be explicitly stated in the Act that the engagement of a professional engineer will not serve to absolve the employer of such an engineer from legal responsibility for his acts and their consequences; that is to say, that it will not shift such legal responsibility either upon the professional engineer or upon the State.

IV The terms of the Act fail to make adequate provision for its application, in that an examining board of nine members, assisted by one secretary and one stenographer, cannot possibly meet the practical requirements entailed.

V The proposed terms of the Act are objectionable in that:

(a) The members of the examining board (Section 3, Paragraph 6) "shall serve without compensation" and shall only be allowed "expenses actually and necessarily incurred in the performance of their duties." If under the head "necessary expenses" the act is intended to provide for the engagement of a corps of examining specialists, membership on the examining board without compensation would be apt to be viewed as an honorary appointment involving comparatively slight demands on time and energy. If, on the other hand, the examining board should be expected personally to prepare the examination questions and to examine the examination papers, it is wholly unreasonable to expect engineers of high professional standing to render such service to the state without liberal remuneration. Moreover, it is both undignified and unfair for a great state to expect individuals

to render professional services of such a high grade and such great volume at their personal sacrifice. In this respect, too, the conditions in engineering under an adequate interpretation of the purposes of this Act, differ radically from those in law and medicine. The examinations for licenses to practice the latter professions are designed to test inexperienced graduates of law and medical colleges as to their knowledge of such fundamental principles as they may fairly be expected to command. The conduct of such examinations is a comparatively simple matter. In engineering, on the other hand, examinations designed to accomplish the expressed purposes of this Act would have to be of a specialized kind and of advanced grade to test the fitness of the candidates for high professional responsibilities.

(b) No recognition is given to those members of the profession who hold responsible teaching positions, as is the practice of leading national engineering societies. This paragraph should be amended by adding the following sentence, quoted from the constitution of the American Society of Civil Engineers:

The performance of the duties of a professor of engineering in a technical school of a high grade shall be taken as an equivalent to an equal number of years of actual practice.

VI Other features of the proposed Act are fairly open to question and criticism, but it is thought best to limit this report to what may be regarded as its most vital features.

C A Proposed Plan for Accomplishing the Avowed Purpose of the Act in a More Direct and Effective Manner

Assuming that the proposed Act is designed solely as a protection to public interests, as stated in its title, and is not designed to elevate professional standards or to promote the pecuniary interests of the members of the engineering profession, its purpose may be accomplished much more directly and effectively through legislation to the following ends:

(a) To create a State Engineering Department, Board or Commission by which engineering designs which may involve public safety would have to be approved before execution. This Department, Board or Commission should also be charged with the inspection of engineering works during construction, at such intervals and to such extent as may be deemed proper.

(b) To exempt from the supervision of this Department, Board or Commission, work designed and constructed by municipalities and approved corporations, such as railroad companies, etc., which are not only legally responsible for, but have an immediate and lasting interest in engineering operations conducted on their behalf and under their supervision.

(c) To require the licensing of county surveyors and other surveyors in independent practice. Fair examinations for surveyors can easily be set, which would serve to exclude incompetent men whose incompetency would be likely to entail property losses and expensive litigation.

(d) To license other classes of technical men whose duties are of a definite and restricted nature in fields where incompetency would be attended by results similar to those just mentioned in connection with surveyors, and in which competency may readily be determined by examinations.

Summarizing, it may be said:

First: That laws under which engineering designs must be officially approved before their execution are in satisfactory operation in certain European countries.

Second: That such laws tend to deter incompetent engineers from preparing designs without competent assistance.

Third: That the Act under review is inadequate as a measure of protection to public interests.

Fourth: That this Act is objectionable in that it will license as "Professional Engineers" men who, neither by experience nor professional attainments, can safely be entrusted with the responsible designing of engineering works. The fact that they have been licensed will tend to invite confidence in their competency on the part of the public to a degree which they would otherwise not command.

Fifth: That in so far as this Act may tend to delay the enactment of statutes of the kind here proposed, its effect would be directly hurtful to public interests.

Respectfully submitted,

John Sterling Deans, Chief Engineer, Phœnix Bridge Co. Richard L. Humphrey, Consulting Engineer. John W. Ledoux, Chief Engineer, American Pipe & Construction Co. H. R. Leonard, Engineer of Bridges & Buildings, Pennsylvania R.R. Co. Henry H. Quimby, Chief Engineer, Dept. of City Transit. S. M. Swaab, Civil Engineer, Keystone State Construction Co. Edward B. Temple, Asst. Chief Engineer, Pennsylvania R.R. Co. Samuel Tobias Wagner, Asst. Engineer, Philadelphia & Reading Ry. Co. Edgar Marburg (Chairman), Professor of Civil Engineering, University of Pennsylvania.

Committee of the Philadelphia Association of Members of the American Society of Civil Engineers appointed to represent that Association in opposition to legislation for licensing professional engineers in the State of Pennsylvania.

NEWS NOTES

A Cave-in in a Sewer Trench in South Bend, Ind., on Oct. 13, caused the death of three men.

Earthquakes Are Reported in Greece, where considerable damage has been done at Thebes, Chalcis and Athens.

A 20-In. Water Main Burst, on Oct. 16, at the corner of Second and Market St., Philadelphia, Penn. Water flooded the basements of several business establishments, and for a time threatened to tie up traffic in the subway.

A Collision Occurred in the New York City Subway on Oct. 17, at the 116th St. station of the Lenox Ave. line. Failure of the brakes of a northbound train caused it to collide with the rear of the train ahead just as the latter was leaving the station. No one was seriously injured, but the insulation of one of the motors was so damaged as to set fire to the car.

Two Sewer Cave-Ins occurred in Toronto, Ont., Oct. 9. A wagon loaded with coal and drawn by a team of horses went through an apparently sound pavement on Margueretta St., falling into a cavity 10 ft. deep over a sewer cave-in. The driver was uninjured but one of the horses was killed. A cave-in during the construction of a new sewer on Swanwish Ave. buried three men on the same day. No one was seriously injured.

A Slide in Culebra Cut, Panama Canal, on Oct. 15, entirely closed the channel about 1500 ft. north of Gold Hill for a distance of several hundred feet. The slide was on the east bank of the canal. It is also reported the bottom of the channel is being forced up on the west side. Dredges were at once set to work, and on Oct. 20 the canal was again opened for traffic, a new channel 150 ft. wide and 40 ft. deep being dredged at record speed.

The 5000-ton Steamship "Metapan" Was Sunk by a collision with the 8000-ton steamship "Iowan," at the entrance to Ambrose Channel, New York Harbor, Oct. 15. The "Metapan" of the United Fruit Co., was inbound from Colon and the West Indies, and the "Iowan", of the Hawaiian-American Line, was outbound. The passengers and crew of the wrecked steamship were saved. The "Metapan" was headed for shore and was beached in 18 ft. of water, a few minutes after the collision.

A Boulder Fell on a Passenger Train on the Denver & Rio Grande R.R., 18 miles east of Grand Junction, Colo., on Oct. 11. The accident occurred at a point where the railway tracks closely parallel the Rio Grande, with cliffs rising high above the river bank. The train was running at high speed when a rock falling from the cliffs struck the coupling between the tender and the smoking car, which parted the train. Before the cars came to a stop a large boulder fell on the smoking car and day coach following, smashing in the roofs and buckling in the steel sides. Some 20 passengers in the two cars escaped injury, while three were killed and 14 were injured. Much of the wreckage was swept into the river.

A Pier Fire on the Delaware River waterfront in Philadelphia, on Oct. 15, destroyed a new pier shed just being completed for the Philadelphia & Reading R.R., causing a loss of about $100,000. The floor of the pier and its foundation were of substantial concrete construction and were uninjured by the fire. The pier shed was a steel structure with wooden mezzanine floors and roof. A small quantity of freight had been placed on the pier, but except for the burning of a few barrels of whiskey the heat of the fire was furnished chiefly by the wooden flooring and roof. The steel structure promptly collapsed on exposure to the flames. The front of the pier facing on the street was of reinforced concrete pierced by large openings. This was not quite completed when the fire occurred, but remained standing, and except for some cracking and spalling was little injured by the flames. Part of the windows in this front wall were of wire glass in steel frames, and this remained intact, no opening through it being made.

The heat of the fire through which this reinforced concrete passed with so little injury was so great as to start fires in storage warehouses protected by fireproof shutters on the opposite side of a 100-ft. street. The heavy granite piers on the front of the building exposed to a much lower degree of heat than the concrete were badly spalled.

An Industrial Welfare Conference is to be held at Harrisburg, Penn., in the State Capitol, Nov. 17-19, under the auspices of the Engineers Society of Pennsylvania and the State Labor Department. A call for the conference has been issued by Dr. John Price Jackson, Dean of Pennsylvania State College and Commissioner of Labor. The conference held a year ago was attended by over 2000 persons. A safety and efficiency exhibition will be held in connection with the conference.

The Rapid Transit Traffic of New York City in the year ending June 30, 1914, has been reported by the Public Service Commission for the first district. The existing subway lines carried 340,413,103 passengers or 1,001,215 per working day. All of the elevated lines together carried 311,473,568. The Second and Ninth Ave. lines made moderate gains; the Third Ave. line gained heavily and reached its limit of carrying capacity. The Sixth Ave. line, including the west-side line above 53d St., showed a slight decrease.

The First Generating Station owned by the Ontario Hydro-Electric Commission was placed in service Oct. 6. This is at Wasdell's Falls on the Severn River, and has 1200 hp. capacity. It is one of several Severn projects, other sites having been purchased from private owners at Big Chute, Swift Current, Port Severn and Eugenia Falls. When they are connected, the system will have an output of 25,000 hp. Heretofore, the Commission has purchased power (notably at Niagara Falls, and transmitted it for distribution and resale by municipalities.

Economies in the Boston, Mass., Health Department—Rather than pay $5000 per annum rent for a more convenient milk laboratory, the city of Boston has increased the salaries of three milk sample collectors $100 per annum, and forced them to walk upstairs to the 11th floor of the new City Hall Annex every night. It is the duty of these collectors to go about the city every night and get samples of milk from delivery wagons. The elevator service to the laboratory ceases at 11 p.m., and legal complications prevent the leaving of the milk samples temporarily in some other place. It was proposed to hire the quarters formerly used on Huntington Ave. for $5000 per annum, so the city fathers figure they have made a saving of $4700 per annum.

Petty Thieving by Drivers of Garbage Wagons is said to be costing the Boston Development & Sanitary Co., which has a 10-year contract to dispose of the wastes of the city of Boston, hundreds of dollars. When depositing the contents of barrels in their wagons, the men take time to pick out such articles as bottles, old metal and shoes and place them in a corner of the wagon. On the way to the receiving station they transfer these articles to bags and dispose of them to junk dealers. The drivers are, in many cases, adding several dollars a week to their incomes in this way, and the company, rightfully claiming that it is entitled to everything in the wastes, is objecting. The Commissioner of Public Works, L. K. Rourke, has suspended a number of drivers for from one to three days as punishment.

The Street Railway Situation at Detroit—In April, 1913, the city voted for municipal ownership of traction lines and the Mayor appointed a board of street-railway commissioners consisting of wealthy automobile manufacturers. They were charged to report on the best methods for building up a new street-railway system or the purchase and development of the old one. This commission appointed E. W. Bemis to make an appraisal of the existing system and he placed a large force at the task. The local company is reported willing to sell. The Bemis report was due Oct. 15 and probably will be made sometime during the month. Pending receipt of this, the commission called in Wm. Barclay Parsons for advice and subsequently retained the firm of Barclay Parsons & Klapp to report on a comprehensive rapid-transit system. The firm's local investigations are going forward under the direction of H. M. Brinkerhoff and this report is expected in February, 1914.

The Production of Explosives in the United States during 1913 has just been published by the U. S. Bureau of Mines. The total was 463,514,881 lb., compared with 489,393,131 lb. for 1912. This production is seggregated as follows: Black powder, 194,146,747 lb.; "high" explosives other than permissible explosives, 241,682,364 lb.; permissible explosives, 27,685,770 lb. These figures represent a decrease of 36,146,622 lb. of black powder and an increase of 7,212,872 lb. of high explosives and 3,055,500 lb. of permissible explosives. In 1902 only 11,300 lb. of permissible explosive was used in coal mining, whereas in 1913 the quantity so used was 21,804,285 lb. The quantity of permissible explosives used in the United States is larger than in a number of foreign countries. In 1912 it represented about 5% of the total quantity of explosives produced, and in 1913 6%. The total amount of explosives used for the production of coal in 1913 was 209,352,938 lb., of which about 10% was of the permissible class as compared with 8% in 1912.

Work on the Austin-San Antonio Post Road, in Texas, was begun Oct. 20. The road is being improved under the supervision of the U. S. Office of Public Roads in accordance with the Act of Congress of Aug. 24, 1912. The total length is nearly 83 miles. Parts of the road have previously been surfaced with gravel. It is proposed to rebuild the road 24 ft. wide between shoulders with a surfaced roadway 15 ft. wide. On portions which have not been graveled before the surfacing is to be 12 in. deep. On the best portions of the old graveled sections the surfacing is to be scarified, reshaped and topped with a new 5-in. (loose measure) wearing course. Other sections will have new wearing courses of from 5 to 12 in. thick. The foundation course may be either adobe rock, or gravel, or any rough gravel which will properly compact. The wearing course will be of selected hard gravel, screened to exclude stones over 2 in. in diameter. All large openings in the roadbed will be bridged with reinforced-concrete structures of not less than 20 ft. clear roadway. Smaller culverts will be of vitrified clay pipe or ingot-iron pipe with concrete headwalls. All wooden structures will be replaced. The estimated cost is nearly $250,000. Local newspapers are rejoicing because the road "will afford employment to hundreds of farmers who will welcome this opportunity to increase their incomes, following a disastrous cotton season."

A Partial Sanitary Reconnaissance Survey of Ohio is being made by the Ohio State Board of Health under the direction of W. H. Dittoe, Chief Engineer. As many of our readers will remember, the board named made an extended investigation of the water-supplies of the state some fifteen years ago, a special investigation of water- and sewage-treatment plants about six years ago and a special study of garbage and refuse disposal some four years ago. The board has also regularly made examinations of the water-supplies and sewerage systems of the state. Mr. Dittoe informs us that these earlier investigations have not taken into account the conditions existing in communities which have not yet provided themselves with the sort of improvements mentioned. Representatives of the board have therefore begun to visit every incorporated community and every public and private institution in the state with a view of determining the existing conditions as regard water-supply, sewerage, and sewage and waste disposal. The work is being done rapidly in order to cover the state in a reasonable length of time, and it is not expected that a complete report based on the information immediately obtained will be published. What is expected is that the board will be able to tabulate for its own information, as well as for the information of those interested, the essential facts relating to all the public and private water-supplies of the incorporated communities and institutions of the state, as well as like information for sewerage and sewage disposal and refuse collection and disposal. It is believed that with this information in a compact form the board will be able to direct an attack upon those conditions in Ohio which are most prejudicial to the health of the state.

The Longest Recorded Drought in Massachusetts was ended when rain began to fall on Oct. 16. The dry period, which began with Aug. 30, lasted for 47 days, during which time the total precipitation amounted to only 0.21 in. at Boston.

The average rainfall at stations on the watershed of the Wachusett River during September, 1914, was 0.15 in. The lowest previous record for any month was 0.48 in. for June, 1912. On the Sudbury River watershed the average rainfall for September, 1914, was 0.29 in., and the lowest previous record was 0.32 in. for September, 1877. At Lake Cochituate the rainfall was 0.27 in. for September, 1914, the lowest previous record having been 0.38 in. for June, 1873. The records have been kept on the Wachusett watershed for 17 years and on the Sudbury watershed and at Lake Cochituate for 50 years.

As no rain fell during the last two days of August and the first 15 days of October, the precipitation record for the month of September is the rainfall for the entire period. It is said that private records show no such long continued drought since 1817.

Unless there are heavy rains during the remainder of the year, 1914 will take its place in the list of exceedingly dry years which began with 1908. The Committee of the New England Water Works Association, which reported on the "Yield of Drainage Areas" at the recent convention in Boston, presented figures on the Wachusett Watershed and called attention to the remarkable fact that the highest yield of any year since 1907 was less than the lowest yield of any one of the preceding 11 years. The importance of this to engineers attempting to estimate the available yield of a watershed is obvious.

PERSONALS

Mr. Richard Blackstone, M. Am. Inst. M. E., Chief Engineer of the Homestake Mining Co., Lead, S. D., has been promoted to be Superintendent, succeeding the late T. J. Grier. Mr. Blackstone is a native of Pennsylvania, where he was born 71 years ago.

Mr. W. Cameron Forbes, of Dedham, Mass., former Governor General of the Philippine Islands, has been appointed Receiver of the Brazil Railway Co., and its subsidiary the Madeira-Mamoré Railway Co., both incorporated under the laws of the State of Maine. Mr. Forbes is a graduate of Harvard University, class of 1892, and from 1897 to 1902 was chief of the financial department of Stone & Webster, Electrical Engineers, Boston, Mass.

Mr. Verne L. Havens, M. Am. Soc. C. E., has been appointed Special Commercial Attaché of the United States Department of Commerce at Santiago, Chile. Mr. Havens was formerly Chief Engineer of the Mexico City Tramways and more recently has been engaged as Consulting Engineer for various Latin-American enterprises. He is the first civil engineer to be appointed Commercial Attaché. The appointment of three other attachés was also made during the past week.

The following city officials have been appointed under the new charter of St. Louis, Mo., which was outlined in our issue of July 9, 1914, p. 104: Messrs. Edward E. Wall, M. Am. Soc. C. E., Water Commissioner of the Department of Public Utilities; A. I. Jacobs, Supervisor of City Lighting of the Department of Public Utilities; Charles M. Talbert, Director of the Department of Streets and Sewers; J. M. Slater, M. Am. Soc. C. E., former Principal Assistant Engineer of the Wabash R.R., Street Commissioner.

Mr. C. Loomis Allen, M. Am. Soc. C. E., President of the Newport News & Old Point Ry. & Electric Co., Vice-President of the Maryland Electric Rys., and President of the Syracuse & Suburban R.R., of Syracuse, N. Y., has been elected President of the American Electric Railway Association. Mr. Allen was born in Syracuse 44 years ago. He attended Alfred and Syracuse Universities and began his engineering experience as an axman with the engineering corps of the Norfolk & Western Ry. As a member of the firm of Mather & Allen, he had charge of much of the civil engineering work in the electrification of the railways in Syracuse from 1892 to 1895. Later he was Chief Engineer and General Manager of the Syracuse street railways.

OBITUARY

William W. Lobdell, President of the Lobdell Car Wheel Co., of Wilmington, Del., died Oct. 10. He was 70 years old.

George A. Gindele, a prominent contractor of Chicago, Ill., died at his home in that city, Oct. 10. He was born in Bavaria in 1844.

Sherlock H. Byron, Superintendent of Buildings of the Public Service Corporation, Newark, N. J., died Oct. 12, at his home in Flushing, N. Y.

Charles Francis Danforth, a civil engineer for many years with the Hecla Architectural Iron Works, of Brooklyn, N. Y., died at his home in Brooklyn, Oct. 9.

Frederic John Henry Rickon, M. Am. Soc. C. E., Civil Engineer and Contractor of San Francisco, Calif., committed suicide at his offices at 1859 Geary St., Oct. 7, by shooting himself with a revolver. He was 60 years old and had for some time suffered from heart trouble. He was a prominent Mason and was President of the board of directors of the Albert Pike Memorial Temple. He is survived by a widow and two sons.

Henry Herbert Yard, M. Am. Inst. M. E., Mining Engineer of the North Carolina Mining Co., San Francisco, Calif., died at his home there, Oct. 8. He was born in Trenton, N. J., 63 years ago. He was educated in the public schools of Trenton and Philadelphia, and in 1869 graduated in mining engineering from the Polytechnic College of Pennsylvania. For several years he was in silver-mining work in Colorado. Returning to the East he was for a number of years a prominent consulting mining engineer of Philadelphia and New York City. Since 1902 he had lived in California.

Oliver S. Lyford, former Vice-President of the Chicago & Eastern Illinois R.R., whose railway experience dated back to 1846, died Oct. 12, at the home of his son in Chicago, Ill. He was born in Maine, June 19, 1823. At 23 years of age he entered the service of the old Boston & Lowell R.R. as a watchman and assistant baggage master. In 1851 he was a passenger conductor. He then left the Boston & Lowell R.R. for the Erie Ry., being for a few months shop clerk at Dunkirk, N. Y., and then a passenger conductor. In 1863 Mr. Lyford became Division Superintendent of the Atlantic & Great Western Ry., and later held the same position with the Erie Ry. Subsequently he was General Superintendent of the Hannibal & St. Joseph R.R., of the Kansas Pacific Ry., and in 1878 first became connected with the Chicago & Eastern Illinois as Superintendent. He was Vice-President from 1890 until about a year ago, when he retired at the age of 90 years, after a total railway service of 67 years.

ENGINEERING SOCIETIES

COMING MEETINGS

NORTHWESTERN ROAD CONGRESS.
Oct. 28-31. Congress at Milwaukee, Wis. Headquarters, 71, Sentinel Bldg., Milwaukee, Wis.

AMERICAN ROAD CONGRESS.
Nov. 9-14. Atlanta, Ga. Permanent headquarters, American Highway Association, Colorado Building, Washington, D. C.

INDUSTRIAL WELFARE AND EFFICIENCY CONFERENCE.
Nov. 16-20. Conference at Harrisburg, Penn. Secy., R. M. Pennock, Harrisburg, Penn.

NATIONAL MUNICIPAL LEAGUE.
Nov. 17-21. Annual meeting at Baltimore, Md. Secy., Clinton Rogers Woodruff, North American Building, Philadelphia, Penn.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
Nov. 19-20. Annual meeting at Columbus, Ohio. Secy., F. E. Sanborn, Ohio State University, Columbus, Ohio.

AMERICAN SOCIETY OF REFRIGERATING ENGINEERS.
Nov. 30-Dec. 1. Annual meeting at New York. Secy., W. H. Ross, 154 Nassau St., New York City.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Nov. 30-Dec. 5. Annual meeting at Jacksonville, Fla. Secy., Prof. Selskar M. Gunn, Massachusetts Institute of Technology, Boston, Mass.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Dec. 1-4. Annual meeting at New York City. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

AMERICAN INSTITUTE OF ARCHITECTS.
Dec. 2-4. Convention at Washington, D. C. Secy., D. Knickbacker Boyd, The Octagon, Washington, D. C.

AMERICAN MINING CONGRESS.
Dec. 7-11. Annual meeting at Phœnix, Ariz. Secy., J. F. Callbreath, Denver, Colo.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Dec. 9-12. Convention at Richmond, Va. Secy., C. A. Prosser, 140 W. 42d St., New York City.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Dec. 10-11. Annual meeting at New York City. Secy., D. H. Cox, 29 W. 39th St., New York City.

AMERICAN ROAD BUILDERS' ASSOCIATION.
Dec. 14-18. Convention at Chicago, Ill. Secy., E. L. Powers, 150 Nassau St., New York City.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS.
Dec. 28-30. Annual meeting at Madison, Wis. Secy., F. M. White, Madison, Wis.

Society for the Promotion of Engineering Education—The council of the society has decided to hold the 1915 meeting at Iowa State College, Ames, Iowa, June 22-25. Dean Marston, of Iowa State College, is President of the society.

Albany Society of Civil Engineers—The regular annual meeting and election of officers will be held in Albany, Oct. 27. Richard W. Sherman will address the society on "Engineers from the Contractors' View Point." The Secretary is Ernest G. Rayner, Barge Canal Office, Albany, N. Y.

American Society of Civil Engineers—The nominations of officers for 1915 are announced as follows: For President, Charles D. Marx, Professor of civil engineering at Leland Stanford, Jr., University; for Vice-Presidents, Clemens Herschel, Consulting Engineer, New York City; Daniel Bontecou, Consulting Engineer, Kansas City, Mo.; for Treasurer, Lincoln Bush, Consulting Engineer, New York City; for Directors, Frederic Molitor, Consulting Engineer, New York City; John V. Davies, Consulting Engineer, New York City; J. E. Greiner, Consulting Engineer, Baltimore, Md.; John B. Hawley, Consulting Engineer, Fort Worth, Tex.; John F. Coleman, Consulting Engineer, New Orleans, La.; Herbert S. Crocker, Consulting Engineer, Denver, Colo.